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# STRUCTURAL ENGINEERING

## FUNDAMENTAL PROPERTIES OF MATERIALS



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V. 2  
BY

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## PREFACE

This volume is the second of the writer's work on Structural Engineering, which work is intended to cover the parts of the subject most important for the engineer and especially for the engineering students in technical schools.

It was originally intended that the present volume should form the second part of the volume on the Strength of Materials, with which it is closely connected; but in order to avoid making the first volume too bulky, this part is published separately. It does not deal in detail with processes of manufacture, which should be studied, as far as necessary, in the more specialized books herein referred to; but is intended to give the fundamental properties of the principal materials, which should be familiar to the engineer. Thus it does not overlap or replace the detailed works on Materials, but brings together logically, and it is hoped clearly, the fundamental properties, the constitution and physical structure, the importance and effect of various ingredients, the effect of different treatments, and the significance of the specifications used to secure desired properties in the material to be employed.

The book is intended to be used in connection with the previous volume on the Strength of Materials, and the needs of constructing engineers have been mainly considered. The point of view of the practical engineer has been emphasized throughout, as it was in the earlier volume. However incomplete this work, it may be of some interest to the student of the subject to see what portions of it the writer, who has done as much practical engineering work as teaching, considers most important for the constructing engineer to know.

It is hoped that this volume will be found a suitable continuation of the first volume of the series. The third and fourth volumes, dealing respectively with the Theory and Design of Elementary Structures and with structures of more complicated character, are well under way.

The writer's thanks are due to various sources for cuts, tables

and miscellaneous information. Acknowledgments have been made in the appropriate places.

The writer is particularly indebted to his colleague, Professor Albert Sauveur, who has read the chapters dealing with metals; and also to his colleague, Professor G. P. Baxter, who has read the chapter dealing with cement and concrete.

The writer will be grateful to any reader who will call his attention to such typographical errors as may have escaped correction, or to any errors of statement into which the writer may have fallen.

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“Read not to contradict and confute, nor to believe and take for granted, nor to find talk and discourse, but to weigh and consider.”

—BACON.

# STRUCTURAL ENGINEERING

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## FUNDAMENTAL PROPERTIES OF MATERIALS

### CHAPTER I

#### INTRODUCTION

##### MATERIALS USED IN STRUCTURES

**1. Importance of the Subject.**—The structural engineer, being a designer and constructor, must use materials. He must, therefore, be familiar with their methods of manufacture, properties, and strength; otherwise, he cannot intelligently select proper materials for his purposes, understand the reasons for current practice or specifications, frame his own specifications intelligently if they have not been standardized, or test the material so as to satisfy himself that it meets his requirements.

The subject is a large one. A man may spend his life in a study of one of the materials alone, like wood, concrete, or steel, if he goes into it thoroughly. It will not be attempted here to treat the subject extensively; but it is thought desirable to give a short statement of the most important fundamental facts and principles, so that the reader may understand the reasons for certain designs and specifications. References will also be given to works on the subject which the structural engineer should study.

**2. References.**—There are exhaustive works on each of the principal materials used, and some works which discuss all of them in a thorough manner. Among these works of a general character may be mentioned the following:

(1) Johnson's "Materials of Construction," fifth edition, rewritten by M. O. Withey and James Aston. This is a wonderfully comprehensive work, treating of the manufacture, properties, and strength of most of the materials of construction. Published by John Wiley & Sons, Inc., 1919.

(2) Mills: "Materials of Construction," second edition, edited by H. W. Hayward. This is a briefer but excellent work treating of all usual materials. Published by John Wiley & Sons, Inc., 1922.

(3) Upton: "The Structure and Properties of the More Common Materials of Construction." This is a book of different character from the first two, dealing almost entirely with metals, with an excellent short chapter on cement. It does not treat methods of manufacture, but discusses clearly and thoroughly the properties and the behaviour under tests, and the structure and treatment of steel. Published by John Wiley & Sons, Inc., 1916.

(4) "The Metallurgy of Iron and Steel," by Bradley Stoughton, is an excellent work, with many references for more detailed study. It should be mastered by the engineer before studying the other works dealing with all kinds of materials. McGraw-Hill Book Co. Inc., 3rd. ed., 1923.

(5) Moore: "Materials of Engineering," is a smaller book, giving an excellent brief treatment of the subject. McGraw-Hill Book Co. Inc., 1922.

All of these works should be read by the structural engineer.

Other excellent books might be mentioned, such as:

(6) "The Making, Shaping and Treating of Steel," by J. M. Camp and C. B. Francis, published by the Carnegie Steel Co., 1919.

(7) Rosenhain: "Elements of Physical Metallurgy." Constable & Co. Ltd., 2nd. ed., 1922.

(8) Unwin: "The Testing of Materials of Construction." Longmans, Green & Co., 3rd. ed., 1910.

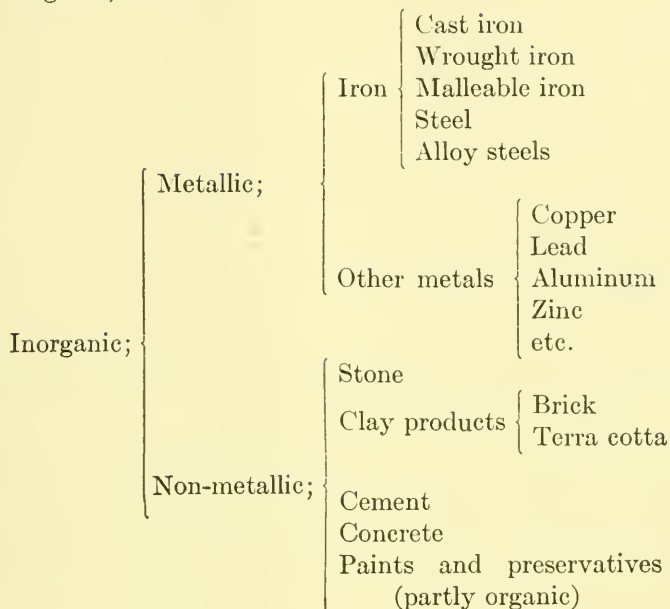
(9) Durand-Claye: "Chimie appliquée à l'art de l'ingenieur." Baudry, 1897.

(10) Martens: "Handbook of Testing Materials," translated by Gus C. Henning, 1899, John Wiley & Sons.

(11) Batson and Hyde: "Mechanical Testing," E. P. Dutton & Co., 1922.

**3. Classification.**—The materials used may be classified as follows:

Organic; wood



4. Specifications for materials have been largely standardized, by the American Society for Testing Materials (A.S.T.M.), the American Concrete Institute (A.C.I.), the American Society of Civil Engineers (A.S.C.E.), etc.; and structural engineers generally rely on the standard specifications, and employ a testing engineer or firm to supervise the tests.

5. Materials of the same kind vary greatly in strength and other properties, depending on chemical composition, physical condition, and the treatment to which they have been subjected, such as seasoning in the case of wood, and mechanical or heat treatment in the case of metals. The reader of this part of the present work will doubtless be impressed by the inherent uncertainties surrounding the subject, and the wide variations in strength of material of the same kind. This should only emphasize the importance of the subject, and that the engineer should know the kind of material he wants and how to get it. Good design without proper material may go for naught.

## CHAPTER II

### WOOD

**1. References.**—There are many works dealing with wood, of which only a few will be mentioned, which should be in the library of the structural engineer.

(1) Snow: "Wood and other Organic Structural Materials." This is an excellent and comprehensive work, and contains a very complete bibliography. Published by McGraw-Hill Book Co., Inc.

(2) Betts: "Timber, Its Strength, Seasoning, and Grading." This book, by an officer of the U. S. Forest Service, is also excellent, dealing largely with the strength of timber. Published by McGraw-Hill Book Co., Inc.

(3) The publications of the U. S. Forest Service treat of every phase of the subject. Snow's bibliography covers them completely.

(4) Hough: "Handbook of the Trees of the Northern States and Canada." This is an excellent reference book, profusely illustrated.

Both Johnson and Mills have excellent chapters on timber.

**2.** Wood is the solid part of trees. The term "wood" includes all woody tissue, even that which is not suitable for construction. "Wood that is suitable, although not necessarily ready, for construction, is 'timber'; and wood that is not only suitable, but also ready for construction, is 'lumber.' The word timber may thus include living trees in the forest, as well as logs and shaped pieces; whereas, lumber refers only to boards, planks, beams, and other sawn pieces of limited sizes."

. Wood is a fibrous material, consisting generally of partly interlaced cells or hollow tubes running lengthwise of the trunk or branch, more or less permeated by water, resins, coloring matter, etc.

For the engineer in the temperate zone there are but two families of trees to be considered, namely, the *Coniferae*, often known as softwoods or evergreens, and the *Dicotyledons*, generally known as hardwoods or deciduous trees.

The coniferous trees comprise the pines, spruces, firs, and cedars, and are the principal source of wood for engineering construction. Their seeds are naked, borne in cones, and the leaves are needle-shaped, though in some cases broader than this name



would indicate. While often called softwoods, their wood is sometimes hard—harder than that of some of the so-called hardwoods—and the leaves of some drop off each year, and so they are not all evergreens.

The other family might best be called *broadleaf trees*,<sup>1</sup> for the wood of some is soft, and the leaves of some are persistent. To this family belong the oaks, beeches, birches, maples, hickories, etc. Their seeds are enclosed in seed vessels.

3. Both of these kinds of trees grow by the addition, each year, of an outside layer or ring of cells, between the old wood and the bark, and by a budding or telescopic extension at the ends of the limbs. A cross-section plainly shows these annual rings, surrounding the pitch center, and by counting them the age of the tree may be estimated.<sup>2</sup> The annual rings vary in thickness, and are not always uniform in the same ring. There are also horizontal cells, forming what are called medullary or pith rays, which are seen in a section, radiating from the center. A nail driven into a trunk remains at the same height as the tree grows, but it may be covered by new wood.

The cells toward the outside of the trunk are more or less filled with sap, which passes upward from the roots through the outer layers to the foliage, is there changed, and passes down through the inner bark to the growing parts. The sap circulates during most of the year. The outer or newer part of the trunk is therefore called *sapwood*, and the inner part *heartwood*, which is modified sapwood. In the heartwood, the cells become filled with gums and inert materials, and it is heavier when dry and of a different and darker color than sapwood.<sup>3</sup> It takes no part in the processes of growth, and is thus virtually dead. The tree can survive even if the heartwood is largely decayed. In some trees the section appears to be almost wholly heartwood and the change of sapwood to heartwood progresses rapidly. In other trees the change is slow.

Each annual ring consists of *springwood*, which grows rapidly in the spring, and *summerwood*, which grows more slowly and is darker colored, and easily distinguished from the spring wood of

<sup>1</sup> SNOW.

<sup>2</sup> Palms, bamboos, and some other tropical plants are not outside growers (Exogens), but inside growers (Endogens), the new woody fiber being irregularly intermingled with the old.

<sup>3</sup> When green, the sapwood contains more water than the heartwood, and is heavier.

the succeeding year. The greater the proportion of summer-wood, the greater the weight and strength.

Heartwood is generally preferred to sapwood, and is considered stronger. It is no doubt more resistant to decay than sapwood, because it is more penetrated by gums and resins, but as regards strength the Forest Service states<sup>1</sup> that, other things equal, sapwood is as strong as heartwood, except from old, overmature trees. Owing to the gums and resins in heartwood, it is less easily treated with preservatives.

**4. Moisture.**<sup>2</sup>—Wood contains considerable moisture when freshly cut, and as it dries the moisture evaporates and the wood shrinks. Aside from the water which is chemically combined with the woody fiber, the moisture in wood is of two kinds; (1) that which is contained in the open spaces in the cells, the cells being hollow tubes; and (2) that which is absorbed by the cell walls, filling the smaller intermolecular spaces. The removal of moisture of the first kind has no effect on the physical properties of the timber except to diminish its weight; the removal of the second kind causes shrinkage and increases the strength, the cell walls becoming thinner. If we should start with wood as dry as possible, the strength would decrease as the moisture of the second kind increased, up to the point when the cell walls were saturated, after which increased moisture would cause no change in strength, and no more swelling. Kiln-dry wood contains about 8 per cent of moisture in all species, and at the fiber saturation point the percentage is about as follows:

#### FIBER SATURATION POINT OF WOOD

*Percentage of Moisture. (U. S. Forest Service, Circular 108)*

Kind of wood	Percentage of moisture at fiber saturation point
Longleaf pine.....	24-26
Red spruce.....	29-35
Chestnut.....	24-28
Loblolly pine.....	23-26
White ash.....	19-23
Red gum.....	25
Douglas fir.....	23
Norway pine.....	26-34
Tamarack.....	27-33

<sup>1</sup> *Bull.* 108, on "Strength of Structural Timber."

<sup>2</sup> U. S. Forest Service, *Cir.* 108 and *Bull.* 70.

If, in addition to the cell walls being saturated, the cells themselves are filled with water, or partly filled, the percentage of moisture will of course exceed these figures.

Tests of strength of small specimens of woods, even of the same species, show large variations in the results. The main cause of these variations is differences in per cent of moisture. The strength when the cell walls are saturated may be only one-third to one-fourth of the strength when the wood is absolutely dry.<sup>1</sup> In order to make results comparable, there should be a standard per cent of moisture. Stiffness is also reduced by moisture, but not so much as strength.

Besides influencing strength, stiffness, and weight, the removal of moisture causes shrinkage, as already mentioned, and often produces defects. The shrinkage longitudinally with the grain is so small as to be negligible, only a few tenths of 1 per cent. The transverse shrinkage is less in a radial direction than tangentially

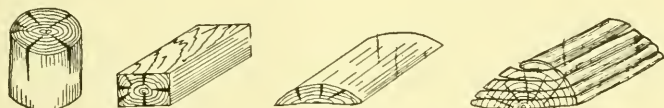


FIG. 1.—Effect of shrinkage of timber.

along the annual rings. This is partly due to the structure, and also to the fact that the outer layers contain the most moisture in a green state, as the sap circulates mostly in these layers. The moisture decreases towards the heart, and there is much less in the heartwood than in the sapwood. There may be little or none in the heartwood, depending on the kind of wood. Longleaf pine has very little moisture in the heartwood, where the cells are filled with gums, pigments, and resins, while the green sapwood is very wet. The shrinkage tangential to the annual rings is generally about twice that in a radial direction. The actual amount of shrinkage, from the green to the oven-dry condition, varies from about 2 to 8 per cent radially (for the conifers 3 or 4 per cent), and from about 5 to 15 per cent tangentially (for the conifers 6 to 8 per cent). A log cut into four parts by two cuts at right angles through the pith will have the four right angles changed to acute angles by the shrinkage; and a rectangular plank, cut tangentially, will become curved, convex

<sup>1</sup> BETTS, pp. 28-29.

toward the heart (Fig. 1); if the grain is not straight, it may also become twisted, or one end of the board may curve more than the other; if the grain is spiral, the twist may be greater.

The cell walls are thicker in summerwood than in springwood, and the former almost always shrinks more than the latter. For the same reason, dense wood shrinks more than light wood of the same species. J. A. Newlin computes that the per cent of radial shrinkage is 9.5 times the specific gravity, and of tangential shrinkage 16.5 times the specific gravity.

**5. The Unequal Shrinkage of Wood Produces Internal Stresses.**—The outer layers, containing the most moisture, dry first and tend to shrink, but their shrinkage is opposed by the inner layers which are still moist. This produces a circumferential tension in the outer layers, and a radial compression between the outer and inner layers. As the tensile strength circumferentially is very small (see Art. 11) radial cracks, called “checks,” are often produced, extending inward radially from the outside (Fig. 1). If these relieve the internal stress, then as the inner layers shrink they tend to separate from the outside layers, and “shakes” or cracks consisting of separation between the annual rings, may be formed, as well as internal radial checks, not visible on the surface. If the outside dries too rapidly and becomes set in a partially dry condition while the inside is still moist, “casehardening” is produced, and the interior, as it dries, will tend to produce shakes and inside checks. The ends of a stick dry more quickly than the parts toward the center of the length, and this often produces radial checks at the ends. If a plank is cut just to one side of the pith, but near it, the effect of shrinkage and wear is often to cause warping, and the inner layers to peel off; such a plank should be laid with the heart side down.

**6. Seasoning**<sup>1</sup> is drying the wood, and may be done by piling it in the open air, or by drying it in an oven. In air seasoning, the wood should be piled so that the air will circulate around it, and so that rainwater will be quickly drained off. Seasoning should be done in a manner to prevent, as far as possible, the formation of checks and shakes. It should not be dried too rapidly, but as uniformly as possible. The moisture should not be evaporated from the outside faster than it can be brought to the outside from the interior.

<sup>1</sup> See BETTS, chap. V.

Air-dried wood contains about 12 per cent of moisture; kiln-dried wood 8 per cent or less, depending on circumstances. Seasoned timber should be used for all interior work, such as floors, where shrinkage would produce objectionable cracks.

**7. Defects of Timber.**—These are knots, checks, shakes, wane, rot, pitch pockets, cross-grain. Scarcely any timber, except small pieces, is free from defects, and it is therefore necessary to classify them and to specify how far they are allowable. They have been defined as follows by the A.S.T.M.

*Knots* are caused by growing branches, or by dead branches adhering to the trunk during growth. If a branch dies and drops off while the tree is small, there will be a small knot near the heart. Knots may be of various sizes and shapes, and may be sound, loose, rotten, encased, or pith.

A *pin-knot* is a sound knot not over  $\frac{1}{2}$  inch in average diameter.

A *standard knot* is a sound knot  $\frac{1}{2}$  to  $1\frac{1}{2}$  inches in average diameter; and a *large knot* is one over  $1\frac{1}{2}$  inches.

A *round knot* is oval or circular; a *spike knot* one which is sawn in a lengthwise direction.

A *sound-knot* is solid and as hard as the wood, and fixed in position; a *loose knot* is one not firmly fixed in place; a *rotten knot* is not as hard as the surrounding wood; a *pith knot* is sound, with a pith hole not over  $\frac{1}{4}$  inch in diameter at the center; an *encased knot* is surrounded wholly or partly by bark or pitch.

The continuity of the wood fibers is of course interrupted or deflected by a knot.

*Checks* are radial cracks often caused by seasoning. A through check extends from one face to the opposite face.

*Shakes* are separations between the annual rings, often called "ring shakes" or "cup shakes." They may be due to seasoning, or to the stresses caused by bending and twisting by the wind. A through shake extends from one face to an adjoining or opposite face. Sometimes a radial check near the heart is called a heart shake.

Checks and shakes may exist in the interior of a stick, invisible from the outside. If on the outside, they may be invisible when the timber is green.

*Wane* is irregularity on the edge of a piece.

*Rot* is any form of decay.

A *pitch pockets* is an opening between the annual rings containing pitch or sometimes bark; if not over  $\frac{1}{8}$  inch wide it is small,



if not over  $\frac{3}{8}$  inch wide or 3 inches long it is standard, if larger than this it is classed as large.

*Cross-grain* is where the fibers are not parallel to the edges. It may be due to the fact that the piece was not sawed parallel to the annual rings, so that the latter run diagonally across the stick. *Spiral grain* is caused by the fibers growing spirally around the pith instead of parallel to it.

**8. Grading Rules and Specifications.**<sup>1</sup>—The manufacturers of the various kinds of lumber, through their associations, have formulated so-called "grading rules," which divide lumber into various grades, and specify what defects are allowable in each grade. The engineer who writes a specification desires material of a quality conformable to his needs and to the stresses which he has deemed it proper to allow in his designs. He must, therefore, be familiar with the grading rules, in order to know which of the grades will satisfy his requirements. There is at present much confusion as to these rules, owing to the large number of lumber associations, railroad companies, and engineering societies which have formulated them. A proper grading rule and specification should be such that no dangerous piece would be accepted under it, and yet such that no piece would be excluded that really belongs in the grade specified. Grading rules and specifications should go together, but the former are prepared as a rule by the manufacturers and the latter by the users. Perfect timber cannot be obtained except at prohibitive cost, and many defects are quite innocuous if the timber is properly used. The grade necessary, and the defects allowable, will depend upon circumstances and the use to be made of the lumber. Many grading rules do not properly classify timbers according to structural strength, though they may be sufficient for woodworking industries or where appearance is the main element.

The reader is referred to the authorities quoted for full information as to grading rules. Betts gives a number of specifications, from which the following are taken:

**CHESAPEAKE AND OHIO RAILWAY COMPANY**  
**SPECIFICATIONS FOR LONGLEAF YELLOW PINE**

All lumber must be cut from living timber of good quality, and must be free from splits, shakes, loose or decayed knots, or defects which impair

<sup>1</sup> See BETTS, SNOW, or JOHNSON. Also U. S. Forest Service Bull. 71, "Rules and Specifications for Grading of Lumber."



its durability. It shall be well manufactured to the size ordered, and must be of longleaf virgin pine; no shortleaf yellow pine, bull pine, or loblolly pine will be accepted under this specification.

*Y.P.*<sup>1</sup> 1

Bridge guard rails, platform joists, signal masts, bumper posts, mail-crane posts, trestle posts, cattle guards, semaphore posts, sills and braces, watertank frames, and dock timbers.

All square lumber shall show two-thirds heart on two sides and not less than one-half heart on the other two sides. Other sizes shall show two-thirds heart on faces, and show heart on two-thirds of the length on edges, excepting where the width exceeds the thickness by 3 inches or over; then it shall show heart on the edges for one-half the length.

*Y.P.* 2

Bridge cross-ties, bridge stringers, watertank joists, scale timbers, and trestle caps.

On all square sizes the sap on each corner shall not exceed one-sixth the width of the face; when the width does not exceed the thickness by 3 inches to show one-half the heart on narrow face the entire length; exceeding 3 inches to show heart on narrow face the entire length; sap on wide faces to be measured as on square sizes.

*Y.P.* 3

Boards and plank for flooring.

Boards and plank 9 inches and under wide to have at least one heart face and two-thirds heart on opposite side; boards and planks over 9 inches to have not less than two-thirds heart on both sides.

**BUILDING CODE RECOMMENDED BY THE NATIONAL BOARD OF  
FIRE UNDERWRITERS, NEW YORK, 1915**

**Rules for grading structural timbers of southern yellow  
pine, prepared in cooperation with the United  
States Forest Service**

**GRADE I<sup>2</sup>**

(1) Requirements for Quality of Timber Based Upon Soundness and Density.—(a) Soundness.—Shall contain only sound wood.

(b) Density, as indicated by number of rings and proportion of summerwood.—Shall show on the cross-section an average of not less than one-third summerwood, measured over the third, fourth, and fifth

<sup>1</sup> Y.P. = Yellow pine.

<sup>2</sup> This is practically the same as the "Select Structural" grade adopted by the Southern Pine Association.

inches on a radial line from the pith. Timber averaging less than six annual growth rings per inch shall show an average of not less than one-half summerwood. Contrast in color between summerwood and springwood shall be sharp.

In cases where timbers do not contain the pith, and it is impossible to locate it with any degree of accuracy by curvature of the rings, the inspection shall be made over 3 inches of an approximately radial line, beginning at the edge nearest the pith.

(2) Restrictions on Knots in Beams.—Sound knots over  $1\frac{1}{2}$  inches in diameter, or knots over  $\frac{1}{2}$  inch in diameter which are insecurely attached to the surrounding wood, shall not be permitted in the middle half of the length of narrow or horizontal faces of beams; nor in the middle half of

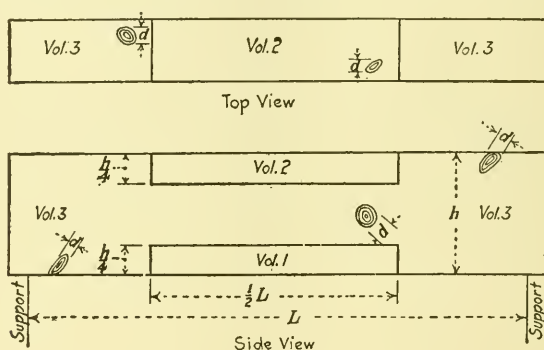


FIG. 2.

the length of the wide or vertical faces within a distance equal to one-fourth their width from the edges (see Fig. 2).<sup>1</sup> No knot shall be permitted within these areas whose diameter exceeds one-fourth the width of the face on which it appears.

The aggregate diameter of all knots within the middle half of the length of any face, shall not exceed the width of that face.

(3) Restrictions on Knots in Columns.—Sound knots having diameters greater than 4 inches or one-third the least dimension of a column, or knots over  $\frac{1}{2}$  inch in diameter which are insecurely attached to the surrounding wood, shall not be permitted.

(4) Restrictions on Shakes and Checks in Beams.—Ring shakes shall not occupy at either end of a timber more than one-fourth the width of green material, nor more than one-third the width for seasoned material. Shakes shall not show on the faces of either green or seasoned timber.

<sup>1</sup> A beam may be divided into three volumes, as shown in Fig. 2. Knots occurring in vol. 1 are the worst, and may almost ruin the wood for strength as a beam. Knots in vol. 2 are less objectionable and in vol. 3 still less objectionable, though much depends on the character of the knot.

Any combination of shakes and checks which would reduce the strength to a greater extent than the ring shakes here allowed, shall not be permitted.

(5) Restrictions on Cross-grain in Beams.—Shall not have diagonal grain with slope greater than one in twenty within the middle half of the length of the beam.

## GRADE II

(6) Requirements for Quality.—Grade II includes timber rejected from Grade I on account of either (a) having less density than required for Grade I; or (b) having more serious defects than are allowed in Grade I.

(a) Timber rejected from Grade I because of deficient density, will be accepted in Grade II provided it meets all the requirements of Grade I, except that in Rule 1, (b), the requirements for one-third summerwood in material having six rings and over per inch, shall be changed to one-fourth; and that the requirement of one-half summerwood in material having less than six rings per inch, shall be changed to one-third.

(b) Timber rejected from Grade I for excess defects will be accepted in Grade II, provided its density conforms to Rule 1, (b), and its defects are limited as follows:

(7) Restrictions on Knots in Beams.—Sound knots over 3 inches in diameter or whose diameter exceeds one-half the width of the face on which they appear, or knots which are insecurely attached to the surrounding wood, whose diameter exceeds  $1\frac{1}{2}$  inches or one-fourth the width of the face on which they appear, shall not be permitted in the middle half of the length of narrow or horizontal faces of beams; nor in the middle half of the length of wide or vertical faces within a distance equal to one-fourth their width from the edges.

The aggregate diameter of all knots within the middle half of the length of any face shall not exceed twice the width of that face.

(8) Restrictions on Knots in Columns.—Sound knots having diameters greater than 6 inches or one-half the least dimension of a column, or knots insecurely attached to the surrounding wood, and having diameters greater than 3 inches or one-fourth the least dimension of a column, shall not be permitted.

(9) Restrictions on Shakes and Checks in Beams.—Ring shakes shall not occupy at either end of a timber more than one-third the width for green material, nor more than one-half the width for seasoned material.

Any combination of shakes and checks which would reduce the strength to a greater extent than the ring shakes here allowed, shall not be permitted.

The so-called "Density Rule," which is the result of investigations by the U. S. Forest Service and the A.S.T.M., and which was

first definitely applied commercially in the inspection of a large shipment of yellow pine for the Panama Canal, is as follows:

#### DENSITY RULE FOR GRADING SOUTHERN HARD PINE

Dense Southern yellow pine shall show on either end an average of at least six annual rings per inch and at least one-third summerwood, or else the greater number of the rings shall show at least one-third summerwood, all as measured over the third, fourth and fifth inches on a radial line from the pith. Wide-ringed material excluded by this rule will be acceptable provided that the amount of summerwood as above measured shall be at least one-half.

The contrast in color between summerwood and springwood shall be sharp, and the summerwood shall be dark in color, except in pieces having considerably above the minimum requirement for summerwood.

In cases where timbers do not contain the pith and it is impossible to locate it with any degree of accuracy, the same inspection shall be made over 3 inches on an approximate radial line beginning at the edge nearest the pith in timbers over 3 inches in thickness, and on the second inch (on the piece) nearest to the pith in timbers 3 inches or less in thickness.

In dimension material containing the pith, but not a 5-inch radial line, which is less than 2 by 8 inches in section or less than 8 inches in width, that does not show over 16 square inches on the cross-section, the inspection shall apply to the second inch from the pith. In larger material that does not show a 5-inch radial line the inspection shall apply to the 3 inches farthest from the pith.

*Sound Southern yellow pine* shall include pieces of Southern pine without any ring or summerwood requirement.

The reader should also consult the following:

Specifications of the A.S.T.M. for Yellow Pine Bridge and Trestle Timbers; and tentative specifications for Structural Douglas Fir.

Specifications of the A.R.E.A. for Southern Yellow Pine Bridge and Trestle Timbers.

Standard Specifications for Grades of Southern Yellow Pine Lumber, issued by the Georgia-Florida Saw Mill Association, 1918. Standard Grading Specifications for Yard Lumber, as recommended by the Dept. of Agriculture: *Department Circular* 296, Oct., 1923.

Those ignorant of the subject may, and sometimes do, commit serious and ludicrous errors in framing specifications. The writer has heard of one specification which read "yellow pine furnished under this schedule must be entirely free from

heart"; and some that called for material "free from sap, knots, wane, and checks, and manufactured from untapped lumber," a requirement probably impossible of fulfillment. Such specifications may be written knowing that they cannot be fulfilled, but with the hope that better lumber will result than under usual requirements; in other words, not expecting to get what is asked for. Such requirements are often disregarded by dealers, who furnish the usual stock. This demoralizes the situation, and often forces honest bidders to resort to the same disregard of the requirements. A specification should only ask for what it is practicable to get, *and should be strictly lived up to*. A bidder who bids on a certain specification hoping to furnish an inferior quality than is called for, and "puts it over," demoralizes the situation.

**9. Kinds of Wood Used in Structures.**<sup>1</sup>—While many different kinds of wood can be used for structures, the requirements of size, strength, durability, etc., are such that the woods used are almost exclusively conifers. Southern yellow or longleaf pine (*Pinus palustris* Mill) is the strongest, stiffest and best of the eastern woods.<sup>2</sup> It formerly covered most of the coastal plain of the southern Atlantic States, but has been largely cut away, and is now becoming scarce. White pine, shortleaf pine, spruce, and other conifers have been largely used in the east.

The great future source of structural timber in the United States is in the forests of the Pacific Coast, where there are large stands of Douglas fir, western hemlock, redwood, and other woods. Of these, Douglas fir is the most suitable for structural purposes.<sup>3</sup> These trees grow to great heights, the tallest specimen recorded being 380 feet high, and they furnish timbers of very large size. Logs that yield timbers 2 feet square and 100 feet long are not uncommon. Western hemlock is also a useful timber, much better and stronger than eastern hemlock. California redwood grows very tall, up to 320 feet high and 35 feet in diameter. It is the oldest of trees, some of them being the oldest living things on earth. It offers unusual resistance to decay and to fire. Sitka spruce was much used during the war for airplane stock. It is light and soft, but strong for its weight. It grows to a height of 296 feet.

<sup>1</sup> See SNOW, Chaps. V, VI, VII, for an excellent treatment.

<sup>2</sup> See *Forest Service Cir.* 164, "Properties and Uses of the Southern Pines."

<sup>3</sup> See "Structural Timber Handbook on Pacific Coast Woods," published by the West Coast Lumbermen's Association, Seattle.



Oak, formerly much used in structures, has given way to the conifers, and is now mostly used for cabinet work. Elm is sometimes used for piling, chestnut for railway ties.

Greenheart,<sup>1</sup> a tree which grows in British Guiana and the West Indies, is stronger and stiffer than any North American wood. Its heartwood is very resistant to decay and to marine wood-borers, very hard and durable. It weighs about 72 pounds per cubic foot when green, and about 57.5 when oven dry. It is hard to work, and liable to split. It is much used for piles and lock gates in northern Europe, and was specified for the sills and fenders of lock gates for the Panama Canal.

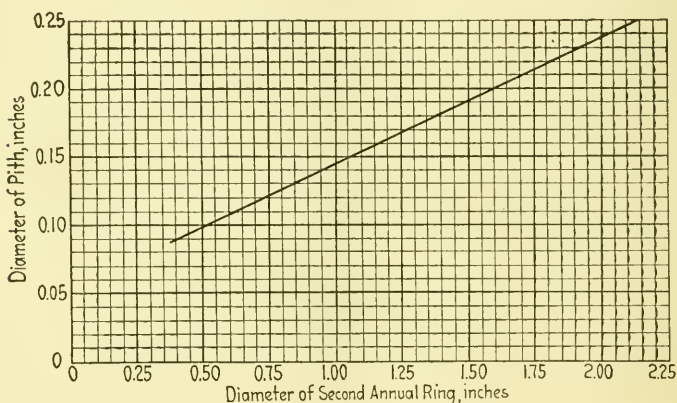


FIG. 3.—Relation of diameter of pith to diameter of second annual ring.

If the point of intersection falls above the diagonal line in the diagram, the specimen is longleaf, or in rare instances slash or pond pine. If it falls below the line, the specimen is not longleaf, but is shortleaf, loblolly, or some other minor southern pine.

Eucalyptus is much used for structures in Australia. Some species of this tree are the tallest, though not the largest, known to man, growing to heights of over 400 feet. The wood is tough and durable.

Bamboo is one of the strongest of all woods, and is much used in China and Japan.

All the kinds of trees in common use for any purpose, and the properties of the woods, are well and fully described by Snow.

10. It is often difficult to distinguish the different kinds of pine woods from one another. Only the expert can distinguish longleaf from shortleaf or loblolly pine. Forest Products Laboratory Technical Note 141 gives a method, if the piece contains the

<sup>1</sup> See ARMSTRONG, A. K., *Engineering Record*, Jan. 29 and Feb. 5, 1916.



pith. This pith, or small, dark core, averages larger in longleaf than in shortleaf or loblolly, being over 0.1 inch in diameter. If the pith in the shortleaf or loblolly is over 0.1 inch in diameter, the diameter of the second annual ring is larger than in longleaf. If the pith in the piece examined is about the size of the lead in an ordinary lead pencil, or smaller, the wood is not longleaf; if it is plainly over 0.1 inch in diameter, and the growth rings surrounding it very narrow, the wood is longleaf. If there is doubt, measure the average diameter of the pith, not including small projections, and the diameter of the second annual ring. Then refer to Fig. 3 and find the point corresponding to these measurements; if it lies above the diagonal straight line, the wood is longleaf; if below, it is shortleaf, loblolly, or some other of the southern pines.

**11. Strength of Timber.**—Most experiments on the strength of timber, particularly those made before 1880, were made on small specimens, say 1 to 4 inches square, in section, of clear timber without defects. Such tests naturally showed greater strength than would result from tests on commercial sizes. Professor G. Lanza, of the Massachusetts Institute of Technology, was the first to show, about 1882, the inapplicability of the results obtained to commercial sizes for practical use and having usual defects, and to make and recommend tests on large specimens taken from ordinary stock. Such tests are the only safe guides for structural design. Tests on small selected pieces may give strengths twice as large as tests on commercial sizes, up to say 6 by 12 inches or even 10 by 16 inches with spans up to 15 feet for transverse tests.

On the other hand, tests on small, clear specimens, free from defects are often chosen, because such tests are the only ones that show the quality of the wood itself, and allow comparisons of the effect of moisture, density, etc. The quality of the wood is an element entirely independent of the presence of defects, which may or may not exist, and are very variable.

The reader is cautioned against comparing values of strength from different sources. Some series of tests have been made without taking any account of moisture or reducing to a common standard. One of the greatest sources of error is in comparing things that are not comparable.

Differences in strength of timber are due mainly to differences in (1) quality of wood, (2) defects, (3) moisture.

The amount of moisture affects greatly the strength given by tests of small perfect specimens, as has been shown. Commercial sizes, such as are used in structures, are difficult to dry thoroughly; if exposed to the weather they would be wet anyway, at times; and the process of drying often produces defects which offset the increase of strength due to drying, as shown by small specimens. Hence, *in structural work, in general, the design should be based on the strength values shown by tests of green timber.* Seasoning cannot be depended on to increase the strength.

The strength of dry timber depends upon (1) the quality of the wood itself, irrespective of defects, and (2) the defects existing, their character, number, location, and condition. *The quality is quite accurately indicated by the dry weight.* As between different kinds of woods (dry), the heavier the wood the stronger it is; and as between different specimens of the same wood, the heavier piece will be the stronger (aside from defects).<sup>1</sup>

The U. S. Forest Products Laboratory suggests the following empirical formulae for the modulus of rupture in the flexure formula, irrespective of species:

Dry timber; modulus of rupture =  $26,200 \times (\text{specific gravity})^{1.20}$

Green timber; modulus of rupture =  $18,500 \times (\text{specific gravity})^{1.20}$

These formulae were derived from tests on small, clear, straight-grained specimens.

The relative weight, and therefore strength, of different pieces of the same species may be judged by the relative proportion of summerwood in the annual rings, summerwood being considerably denser and heavier than springwood.

There is also a certain rate of growth which generally gives the greatest density and strength of a given species. This rate is stated as follows:<sup>2</sup>

VARIETY	RINGS PER INCH	VARIETY	RINGS PER INCH
Douglas fir.....	24	Tamarack.....	20
Shortleaf pine.....	12	Norway pine.....	18
Loblolly pine.....	6	Redwood.....	30
Western larch.....	18	Longleaf pine.....	10
Western hemlock.....	14		

<sup>1</sup> An exception to this rule is found in the timber from the under side of leaning coniferous trees, known as "compression wood," which has very wide rings, and while heavy, is low in strength.

<sup>2</sup> BEGGS, p. 50.

Density, however, is the best reliance, and the "Density Rule," quoted in Art. 8, has been employed for grading southern yellow pine.

*Bleeding pine trees for turpentine* does not affect the strength of the timber. The strength of longleaf pine is independent of the resin content.

The results of tests of strength are very discordant, depending upon moisture, defects, sizes of specimens, and variability in the material itself. The conclusion to be drawn is that the factor of safety should be larger in timber than in more uniform materials, that special care should be used in selecting timber to be used in important structures, that no piece which is seriously defective should be used, and that timber pieces should be so placed that any defects that exist may do the least harm.

*Tensile Strength.*—The ultimate tensile strength of timber along the grain is high, running in some woods up to over 30,000 pounds per square inch. A failure by tension in this direction involves tearing across the fibers. It is difficult to make tension tests, because, even if the specimen is tapered down to a small section at the center, it will be apt to fail by shearing or pulling out, either at the ends where the pull is applied, or at the center if the grain is not exactly straight, as it seldom is. It is difficult to get a pure tensile fracture, and when obtained it is seldom a clean fracture, as in metals, but is much splintered. Tensile tests, however, are seldom made; and fortunately the tensile strength in structures is seldom or never a controlling element, because the strength of the connections of a tensile piece is less than the tensile strength of the piece itself. Its principal application is in the tension fibers of beams. Tensile strength is largely dependent on straightness of grain, knots, and the thickness of the cell walls. It is less dependent on moisture than the other properties are.

The *tensile strength perpendicular to the grain*, which merely involves tearing the fibers from one another, either radially or tangential to the annual rings, is much less than the tensile strength along the grain, rarely exceeding 1,000 pounds per square inch, and in some woods running down to 200 pounds or less. Wood is seldom exposed to tension of this kind, except in some details which will later be pointed out. In the tension fibers of beams that are cross-grained, this kind of tension may exist. The tensile strength across a radial plane is generally less than

across a plane tangent to the annual rings, especially in oaks and hardwoods with large medullary rays.

*Compressive Strength.*—The compressive strength along the grain is considerable, sometimes 8,000 to 10,000 pounds per square inch for short pieces which do not bend. It depends upon the amount of wood fiber in the cross-section, the lateral adhesion of the fibers and their continuity. Failure occurs by crumpling of the cell walls, often showing itself by shearing along



FIG. 4.—Failure of a short wooden block in compression.

a plane inclined somewhat steeper than  $45^\circ$ . A long wooden column may fail by lateral deflection, and a column formula must be used.

Figure 4 shows the failure of a short wooden column in compression.

The *compressive strength across the grain* involves crushing in the cell walls like compressing a hollow tube laterally. Failure, or fracture, does not occur under this kind of stress, and sometimes there is a progressive yielding from the beginning; though

more often there is no appreciable yielding until a certain load is reached, after which the wood crushes continuously. The

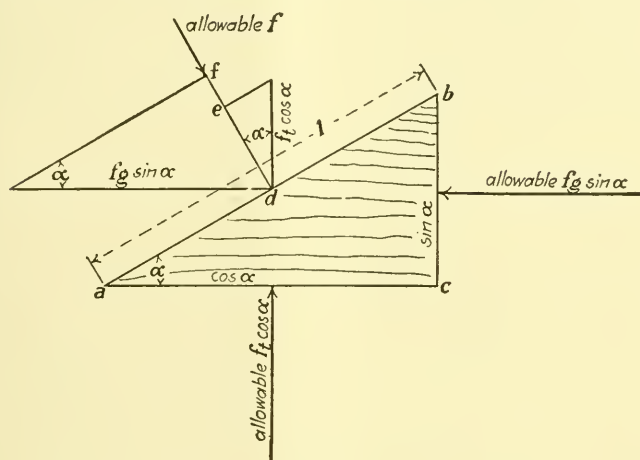


FIG. 5.

strength in this direction is assumed to be that at the elastic limit, or at some specified yielding.

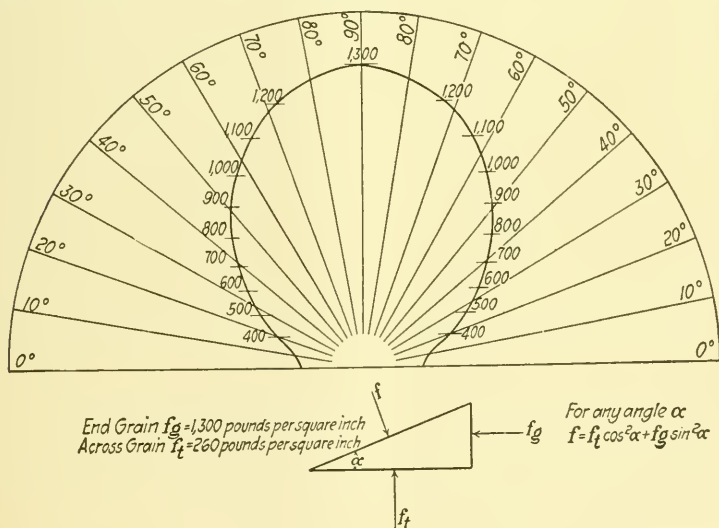


FIG. 6.

If wood is in compression at an angle to the grain, as in Fig. 5, the allowable normal stress may be found as follows: let  $ab = 1$



inch, then  $bc = \sin \alpha$ ,  $ac = \cos \alpha$ ; let the allowable stress along the grain be  $f_g$ , and that across the grain  $f_t$ . Then it seems reasonable that the surface  $ab$  could sustain a stress of  $f_t \cdot ac = f_t \cos \alpha$  perpendicular to  $ac$ , and a stress of  $f_g \cdot bc = f_g \sin \alpha$  perpendicular to  $bc$ . Then the total normal stress on  $ab$ , which is the stress intensity, since  $ab = 1$  inch, is,

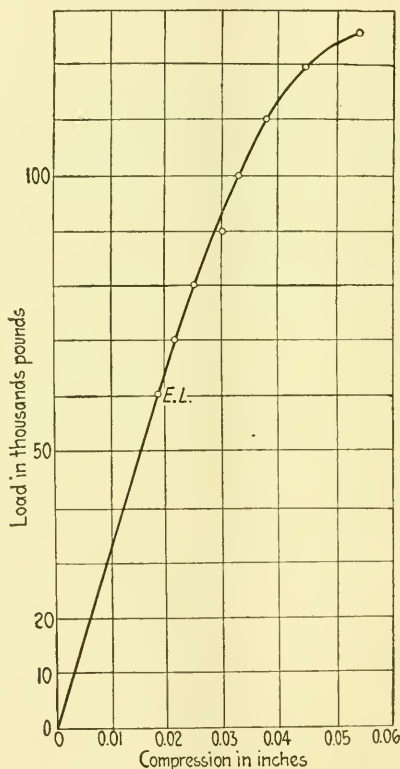


FIG. 7.—Wood in compression parallel to grain. Specimen  $6 \times 6 \times 24$  inches. (Betts, p. 19.)

$$de + df = f_t \cos^2 \alpha + f_g \sin^2 \alpha.$$

Figure 6 gives the allowable stresses for various angles  $\alpha$ , by this formula, namely

$$f = f_t \cos^2 \alpha + f_g \sin^2 \alpha.$$

If the reader thinks that the surface  $ab$  could not safely sustain simultaneously the two forces  $f_t \cos \alpha$  and  $f_g \sin \alpha$ , then he may adopt some empirical rule more satisfying to his instinct.

Figures 7 and 8 show stress-strain diagrams for wood in compression, taken from *Forest Service Bull.* 108. These show a quite well-defined elastic limit, but often the line is more curved, and the limit is not well defined. The initial portion of the diagram, however, is generally nearly straight, and the elastic limit can be approximated to as in Figs. 7 and 8. A wooden piece,

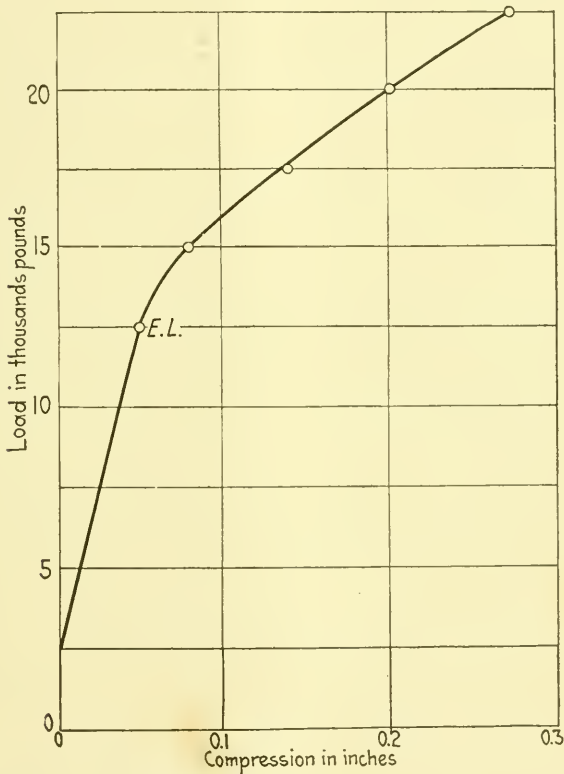


FIG. 8. Wood in compression across the grain. Area  $8 \times 5\frac{3}{16}$  inches. (Betts, p. 20.)

in which the elastic limit is much below the ultimate breaking strength, will give warning before it fails, which is advantageous.

*Shearing Strength.*—This is very different according as it is along or across the fibers. The latter, which involves shearing off a fiber crosswise, is large; but since this shear cannot exist without the simultaneous existence of a shear of equal intensity along the fiber, it is of no importance structurally. The shearing



strength along the fibers, which merely involves sliding one layer of fibers on another and overcoming the cohesion between them, is very small, and acts along a plane in which there may be a check or shake. This shearing strength will therefore be much greater in small selected specimens in which an area of sound wood a few inches square is sheared off, then it will be in large sticks, in which defects may exist. The values found by tests of small specimens are sometimes three or four times those found in beams which, under transverse loads, fail by longitudinal shearing at the end.

*Transverse Strength.*—This is a combination of tension, compression and shearing, and as the tensile strength is greater than the compressive strength, failure will generally occur by compression or shearing, unless there are defects, such as knots or cross-grain, which cause it to fail by tension. The modulus of rupture, as computed, does not represent a stress, because it is computed on assumptions which are not valid up to rupture. It is a purely fictitious stress, and will always exceed the real compressive stress, for reasons that are explained in the chapter on Flexure, in Vol. I. Nevertheless, it serves as a basis of comparison, and, with a proper factor of safety, may be used in designing.

The stress distribution in a wooden beam is not planar, due to lack of homogeneity and difference of elasticity in tension and in compression. The neutral axis in pure flexure does not go through the center of gravity.

**12. Effect of Defects on Strength.**—Checks and shakes may obviously reduce or entirely destroy the resistance to shearing. Judgment and a knowledge of existing stresses are required to use properly timbers with serious checks or shakes.

Cross-grain or spiral grain may be a serious defect in tension or bending, but less serious in compression or shearing. Cross-grain on the tension side of a beam may cause failure. In using Sitka spruce for airplanes, the specification required that no spiral-grained wood should be accepted with over 1 inch twist in a length of 20 inches.

Knots may seriously impair or entirely destroy the strength in tension. If sound, they may do little harm in compression or shearing. If loose, they impair the strength in every way. A beam should, if possible, have no large knots or irregular grain on the tension side near the center of the span. A small knot in

one location, as near the edge of the tension side of a beam, may be more injurious than a large knot on the compression side.

**13. Effect of Time on Strength of Timber.**—The figures generally, if not always, given for strength of timber represent the ultimate strength obtained by one application of the load. In almost all cases a smaller load, if allowed to remain for weeks or months, would ultimately cause failure. It is generally best to assume that the strength under a permanent load is not over one-half that given by a short-time test; and in computing deflection to assume the modulus of elasticity one-half the value given by short-time tests.

**14. Tables of Strength.**—Table I gives data regarding the strength of some timbers, taken largely from more extensive tables given by Betts, and in *Bull.* 556, U. S. Forest Service. These figures are from tests on small, clear specimens, and therefore give the strength independent of defects.

Table II, from Betts gives figures for green structural timbers as compared with those for small clear specimens.

*Bulletin* 556 of the Forest Service gives approximate figures for the percentage of change in the various strength figures and other properties, due to changes in moisture and specific gravity.

**15. Weight.**—Table I gives the weight per cubic foot of the different species. It is a curious fact that the weight of the wood fiber (cellulose) is almost the same for different species of North American woods, the specific gravity of all species being about 1.6; so that no wood would float were it not for the air in the cells, walls, and intercellular spaces. "Accurate determinations by the Forest Products Laboratory on seven species of wood, including both hardwood and coniferous species, showed a range of only about 4.5 per cent. in the density of the wood substance."<sup>1</sup> The greater the specific gravity of a given piece of dry wood, therefore, the more wood substance there is in it, and the greater the strength, normally.

The lightest wood is Balsa, which grows in Central America and the West Indies, and weighs but 7 pounds per cubic foot.<sup>2</sup> The heaviest is black ironwood, which weighs about 81 pounds per cubic foot.

It is common, for bridges, to specify that wood shall be assumed to weigh 4.5 pounds per foot board measure, or 54 pounds per

<sup>1</sup> *Forest Service Bull.* 556. p. 5.

<sup>2</sup> See *Trans. A.S.C.E.*, May, 1916.

TABLE I.—STRENGTH OF TIMBER

Small, clear, green specimens. 2 × 2 inches, 28-inch span in bending. (From Betts and Forest Service Bull. 556)

Kind of wood	Locality	Weights <sup>1</sup> in pounds per cubic foot			Bending modulus of rupture, <sup>3</sup> pounds per square inch	Compression parallel to grain, <sup>4</sup> pounds per square inch	Compression perpendicular to grain at elastic limit, <sup>5</sup> pounds per square inch	Modulus of elasticity <sup>6</sup> in bending, pounds per square inch	Shearing parallel to grain, <sup>7</sup> pounds per square inch	Tension perpendicular to grain, pounds per square inch	Work in bending to elastic limit, inch pounds per cubic inch
		Green	Air <sup>1</sup> dry	Kiln <sup>2</sup> dry							
Beech.....	Pa., Ind.	55	44	42	8,165	3,280	607	1,242,000	1,210	760	0.99
Birch, yellow.....	Pa., Wis.	58	45	43	8,700	3,460	450	1,540,000	1,115	480	0.80
Chestnut.....	Md., Tenn	55	30	29	5,600	2,470	380	930,000	800	430	0.59
Greenheart.....	Demarara, S. A.	72	62	60	18,240	9,910	1,633	2,792,000	1,922	...	1.34
Hickory, pig nut.....	Ohio	64	53	51	12,360	4,760	1,224	1,553,000	1,427	...	1.28
Hickory, shag bark.....	Ohio	64	54	51	10,990	4,360	1,080	1,346,000	1,421	770	1.08
Oak, white.....	La.	65	48	46	8,200	3,600	880	1,260,000	1,248	200	0.86
Douglas fir.....	Wash., Ore.	38	35	34	7,800	3,900	528	1,576,000	909	900	0.65
Douglas fir <sup>2</sup> .....	Mont., Wyo.	35	31	30	6,215	2,375	372	1,276,000	795	350	1.00
Pine, longleaf.....	Miss., La., Fla.	50	42	40	8,654	4,860	583	1,618,000	1,063	290	0.79
Pine, shortleaf.....	Ark., La.	50	38	36	7,960	3,810	475	1,450,000	890	330	0.62
Pine, white.....	Wis.	39	27	26	5,310	2,720	314	1,073,000	614	260	0.56
Redwood.....	Cal.	38	24	23	7,010	4,000	575	1,065,000	764	220	0.84
Spruce, red.....	Tenn.	35	28	27	5,600	2,600	368	1,213,000	863	260	0.58
Tamarack.....	Wis.	47	38	37	7,170	3,480	375	1,236,000	850	260	0.58
Western hemlock.....	Wash.	41	30	29	6,680	2,590	375	1,060,000	850	260	0.58

<sup>1</sup> 12-15 per cent moisture.<sup>2</sup> 8 per cent moisture.<sup>3</sup> Air-dry wood about 50 per cent greater; kiln-dry wood 100 per cent greater.<sup>4</sup> Air-dry wood nearly 100 per cent greater; kiln-dry wood 2 to 3 times as great.<sup>5</sup> Air-dry wood about 1.75 times.<sup>6</sup> Air-dry wood about 25 per cent greater.<sup>7</sup> Air-dry wood about 50 per cent greater; kiln-dry wood greater still: average of radial and tangential.<sup>8</sup> See also Circular Forest Products Laboratory, 1922.<sup>9</sup> The Rocky Mountain type of Douglas Fir is distinctly weaker than the Pacific Coast type.

TABLE II.—AVERAGE STRENGTH OF GREEN STRUCTURAL TIMBERS WITH ORDINARY DEFECTS AND SMALL SPECIMENS  
WITHOUT DEFECTS  
(From Betts)

		Bending			Compression parallel to grain	Compression perpendicular to grain	Shear
		Stress per square inch at elastic limit	Modulus of rupture, pounds per square inch	Modulus of elasticity, 1,000 pounds per square inch	Ultimate crushing strength, pounds per square inch	Stress at elastic limit, pounds per square inch	
Douglas fir.....	Structural sizes	3,968	5,983	1,517	3,495	570	765
	Small specimens	5,227	8,280	1,597	4,030	...	
Loblolly pine.....	Structural sizes	3,040	5,084	1,387	2,940	500	630
	Small specimens	4,100	7,870	1,440	3,240	...	
Longleaf pine.....	Structural sizes	4,015	6,191	1,600	...	495	1,048
	Small specimens	5,270	8,330	1,530	4,285	570	
Red spruce.....	Structural sizes	2,394	3,566	1,180	...	...	758
	Small specimens	3,627	5,900	1,157	2,750	310	
Redwood.....	Structural sizes	3,760	4,472	1,042	3,882	434	742
	Small specimens	4,750	6,980	1,061	3,980	569	
Shortleaf pine.....	Structural sizes	3,237	5,548	1,473	3,435	351	704
	Small specimens	4,350	7,710	1,395	3,570	400	
Western hemlock.....	Structural sizes	3,516	5,296	1,445	3,355	434	630
	Small specimens	4,406	7,294	1,428	3,392	...	
Western larch.....	Structural sizes	3,324	4,948	1,301	3,510	456	700
	Small specimens	4,274	7,251	1,310	3,696	...	
Western yellow pine.....	Structural sizes	2,769	4,560	1,243	2,830	299	651
	Small specimens	3,156	5,831	1,178	2,896	...	
White spruce.....	Structural sizes	2,239	3,288	1,081	...	...	
	Small specimens	3,090	5,185	998	2,370	270	

cubic foot. Such wood is of course likely to be saturated with moisture.

In view of all the foregoing, it must be recognized that timber is a very variable material. The student is advised to read, at this point, Art. 80 of "Structural Details" by Professor H. S. Jacoby, (John Wiley & Sons, Inc., 1910); and to consult the Proc. A.R.E.A. for 1909, pp. 543-565.

**16. Allowable Stresses.**—In deciding upon working stresses, it is necessary to divide the ultimate by a factor of safety, which is variously taken from 4 to 8, and higher still for wood exposed to impact. The factor may also vary according to the kind of stress, and the relative importance of defects.

In bending, a defect such as a knot in the tension side, near the center, or a check near the neutral axis near one end, may make a stick nearly worthless, while its failure might be serious. A factor of 6 in flexure, for steady loads, applied to the ultimate given by short-time tests, which would really be a factor of 3, would not be too large (see Art. 13). This would give for long-leaf pine  $\frac{8,640}{6} = 1,440$  pounds per square inch. A common figure for highway or railway bridges is 1,200 to 1,500.

In compression along the grain a defect would not generally weaken a piece as much as in flexure, and here a factor of 5 might be justified.

In compression across the grain, a defect would have still less influence, and a failure would often cause no serious results, as in bearing on wall plates of bridges. Moreover, there is no ultimate strength for this kind of stress, and the figures given in the table are for the elastic limit. Here the factor of safety may be small, and is sometimes taken as low as 1.5, or the allowed stress is  $\frac{2}{3}$  of the tabular figures, or for yellow pine 390 pounds per square inch. The factor is not generally taken as low as this, but is commonly from 2 to 2.5.

In deciding on the factor of safety, it must be remembered that if, as stated above, the ultimate strength for a steady load is about one-half the value given by short-time tests, then the effect of a suddenly applied load (which is double that of a gradually applied load) is no greater than the effect of a steady load; that is, *the allowance for impact may be much less for wooden structures than for metal structures.*



The factor of safety should also depend upon the use, exposure, expected life, and other circumstances.

The U. S. Forest Products Laboratory gives the following table (III) of working permissible stresses, under different conditions, which merits careful study.<sup>1</sup>

The Committee on Wooden Bridges and Trestles, of the A.R.E.A., recommends the following as safe working stresses (Table IV).

In the "Specifications for Bridges and Subways" by Henry B. Seaman,<sup>2</sup> the following are the allowed static stresses on timber:

Nature of stress	White oak	White pine	Georgia pine	Spruce	Hemlock
Tension, with grain.....	1,500	1,000	1,800	1,200	900
Tension across grain.....	300	75	90	75	
Compression, end bearing.....	2,100	1,600	2,400	1,800	
Compression to 15 diameters...	1,300	1,000	1,500	1,200	1,200
Compression, columns <sup>1</sup> .....	1,600	1,200	1,800	1,400	1,400
Compression across grain.....	750	300	500	300	200
Shear across grain.....	1,500	750	1,900	1,100	900
Shear with grain.....	300	150	200	150	150
Bending, outer fiber.....	1,500	1,000	1,800	1,000	900
Modulus of elasticity.....	1,100,000	1,000,000	1,700,000	1,200,000	900,000

<sup>1</sup> Numerator in the formula  $\frac{P}{A} = \frac{f}{1 + \frac{l^2}{1,000d^2}}$ , but not above the values given in previous

line for 15 diameters.

$l$  = length of column in inches.

$d$  = least diameter in inches.

Committee D-7 of the A.S.T.M., on Timber, makes the following remarks, on pages 389-390 of the *Proceedings of the American Society for Testing Materials*, 1921.

Working Stresses in Structural Timber.—The committee offers the following comments in regard to working stresses in structural timber:

(1) It is well determined that the strength of a particular piece of timber is in a measure determined by the condition under which it is used. From the tests it is seen that increase in moisture decreases the strength of timber, therefore it must be determined, first, in designing a particular structure, what the moisture conditions are.

(2) It is also well determined by the tests that resistance to suddenly applied loads is much greater than to slowly applied or constant loading; therefore the condition of loading will affect the amount of allowable stress.

<sup>1</sup> Circular on Grading Rules and Working Stresses, 1920.

<sup>2</sup> *Trans. A.S.C.E.*, vol. LXXV, p. 313, 1912.

TABLE III.—WORKING STRESSES PERMISSIBLE FOR STRUCTURAL TIMBERS  
(Pounds per square inch)

Species	Bending				Compression					
	Allowable stress in extreme fiber				Allowable stress parallel to grain "short columns"		Allowable stress perpendicular to grain			
	Damp or wet location (docks, piling, and sills)	Outside, not in contact with soil (bridges and open sheds)	Under shelter in a dry location (factories and warehouses)	Allowable horizontal shear stress All locations	Wet location	Outside location	Dry location	Wet location	Outside location	Inside location
Cedar, western red.....	750	800	900	80	650	700	700	125	150	200
Cedar, northern white.....	600	650	750	70	450	500	550	100	140	175
Chestnut.....	700	850	950	90	600	700	800	150	200	300
Cypress.....	900	1,100	1,300	100	800	1,100	1,100	225	250	350
Douglas fir, (No. 1 structural).....	1,100	1,400	1,600	100	900	1,100	1,200	225	250	350
Douglas fir (No. 2 structural).....	900	1,100	1,300	90	800	900	1,000	200	225	300
Douglas fir (Rocky mountain region).....	700	900	1,100	85	700	800	1,000	200	225	300
Fir, balsam.....	600	750	900	70	500	600	700	100	125	150
Gum, red.....	800	900	1,100	100	650	750	800	150	200	300
Hemlock, western.....	900	1,100	1,300	75	800	900	900	200	225	300
Hemlock, eastern.....	800	900	1,000	70	600	700	700	200	225	300
Hickory.....	1,200	1,500	1,900	140	1,000	1,200	1,500	350	400	600
Larch, western.....	900	1,100	1,200	100	800	1,000	1,100	200	275	325
Maple, sugar or hard.....	1,000	1,300	1,500	150	900	1,100	1,200	300	375	500
Maple, silver or soft.....	700	800	1,000	100	600	700	800	200	250	350
Oak, white or red.....	1,000	1,200	1,400	125	800	900	1,000	300	375	500
Pine, southern yellow (dense).....	1,100	1,400	1,600	125	900	1,100	1,200	225	250	350
Pine, southern yellow (sound).....	900	1,100	1,300	105	800	900	1,000	200	225	300
Pine, eastern white.....	750	800	900	85	650	750	750	125	150	250
Pine, western white.....	750	800	900	85	650	750	750	125	150	250
Pine, Norway.....	800	1,000	1,100	85	700	800	800	150	175	300
Redwood.....	800	1,000	1,200	70	750	900	1,000	125	150	250
Spruce, red, white, and Sitka.....	900	900	1,100	85	650	750	800	125	150	250
Spruce, Engelmann.....	500	650	750	70	450	550	600	100	140	175
Tamarack, eastern.....	900	1,100	1,200	95	800	900	1,000	200	225	300

(Revised). U. S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, Wis., Dec. 2, 1919.



(3) Warning of failure of a piece of timber in a structure is usually given a considerable time in advance of actual failure; therefore, when efficient inspection is had this feature gives an element of safety in older structures.

While there is no well-defined "elastic limit" in timber tests, there is in general a region where the proportionality of stress to strain ceases to be constant. It is therefore well to keep stresses well within this limit. Timber, however, does recover from high stresses; therefore its resistance to quick loading.

From the above it will be seen that in order to have a rational design it is necessary to state working stresses of different amounts for different kinds of loadings and exposure of timber. As an example, for dense structural yellow pine, the maximum working stress will be 1,100 pounds per square inch. This is for constant loading and for submerged locations where the timber is constantly wet. In locations in the weather, such as bridges, the allowable working stress for constant loading is 1,400 pounds per square inch. Under cover, where the timber is always dry, the allowable working stress for constant loading is 1,600 pounds per square inch.

From the tests, it is determined that the resistance of timber is approximately proportional to the speed of loading. For constant loading, the stresses above given are proper, but for sudden loading, resulting in 100 per cent impact, the successive loadings being far enough apart so as to allow reasonable recovery of the timber, the allowable stresses may be doubled (not to exceed 2,800 pounds per square inch), the stresses due to this sudden loading being those actually computed from the load with the impact. For other proportions of impact, less than 100 per cent, the allowable working stress may be increased in a ratio equal to the percentage of impact. For dense structural yellow pine, the allowable working stresses would, therefore, be as follows:

1. For wet or submerged locations.....  $1,100 + 1,100 I$  pounds per square inch.
2. For exposed locations  
(bridges).....  $1,400 + 1,400 I$  pounds per square inch.
3. For constantly dry locations  $1,600 + 1,600 I$  pounds per square inch  
where  $I$  is the proportional impact stress.

**17. Decay of Timber.**—Decay of timber is caused by certain fungi or bacteria which gain entrance, develop, and finally destroy the fiber, reducing it to powder, if dry, or to soft pulp, if wet.

The germs gain entrance sometimes from water, sometimes from the air, and their entrance is facilitated by checks. For the development of the germs, certain conditions of temperature and



Redwood.....	5,000	900	800,000	300	80	.....	...	400	150	3,300	900	680	$900 \left(1 - \frac{L}{60d}\right)$
Bald cypress.....	4,800	900	1,150,000	500	120	...	...	340	170	3,900	1,100	830	$1,100 \left(1 - \frac{L}{60d}\right)$
Red cedar.....	4,200	800	800,000	...	...	...	...	470	230	2,800	900	680	$900 \left(1 - \frac{L}{60d}\right)$
White oak.....	5,700	1,100	1,150,000	840	210	270	110	920	450	3,500	1,300	980	$1,300 \left(1 - \frac{L}{60d}\right)$

These unit stresses are for a green condition of timber and are to be used without increasing the live-load stresses for impact.

i Partially air dry.

$L$  = length in inches.

$d$  = least side in inches.

NORE.—The working unit stresses given in this table are intended for railroad bridges and trestles. For highway bridges and trestles the unit stresses may be increased 25 per cent. For buildings and similar structures, in which the timber is protected from the weather and practically free from impact, the unit stresses may be increased 50 per cent. To compute deflection of a beam under long continued loading instead of that when the load is first applied, only 50 per cent of the corresponding modulus of elasticity given in the table is to be employed.

moisture are necessary. Dry wood will not decay, simply because it is dry, nor will wood that is completely submerged all the time or completely saturated. Extremes of wetness or dryness both prevent rot.

"Wet-rot" was erroneously supposed to be due to moisture, and "dry-rot" to dryness, but both are due to the development of organisms of decay, which require different degrees of moisture and temperature.

Sapwood decays more readily than heartwood, as the sap contains compounds that favor decay, while the resins of heartwood resist it. Wood does not decay because it is old. If the organisms do not gain entrance and develop, it may last forever.

Wood, if unprotected and exposed to the weather, may decay in a few years by wet-rot, the alternate soaking with water, and then drying out, favoring decay. Wooden bridges are therefore generally covered by a roof, and exposed wood is painted. Decayed wood can be discovered by its softness, and also to some extent by its color, though there are discolorations that are not caused by decay. It lacks resonance when struck with a hammer, absorbs much water, and sometimes has an odor that indicates decay. The writer, in inspecting wooden railroad structures for many years, carried a cane or rod with a sharp point at the end. Often a wooden stringer would be found with a large part of the corner (sapwood) soft and useless. It is better to remove such rotten wood, for it helps the remaining sound wood to decay. Wherever moisture can collect and be held for quite a time, then drying out, rot is likely to occur. Sills and caps of pile trestles, and connections of wooden structures, if exposed, are likely to go first; and piles in earth decay at and near the ground level. Piles in water decay first near the water level.

On the other hand, in dry places, a timber may look perfectly sound, yet may be a mere shell, the inside decayed to powder. Such timbers will sound hollow when struck with a hammer. The writer found a bridge truss of white pine in which, at one end, three out of four sticks in the chord were so decayed. A long slender auger may be used for boring into suspected timbers.

Timber is sometimes attacked by animals which destroy it. White ants, which attack some timbers in the southern states, may eat away the entire interior, leaving only a shell.

**18. Marine Borers.**—Timber in sea water is subject to attack by the teredo, the limnoria, and the chelura. The teredo enters

the wood, turns at right angles, and bores a tube from  $\frac{1}{4}$  inch to  $1\frac{1}{8}$  inch in diameter, which it lines with a calcareous lining, and in which it lives. Sometimes a pile will be completely perforated by these tubes, and in warm waters, such as the Gulf of Mexico, a 6 inch pile may be destroyed in two or three months. The teredo works as far north as the Gulf of St. Lawrence, though of smaller size there than in warmer waters.

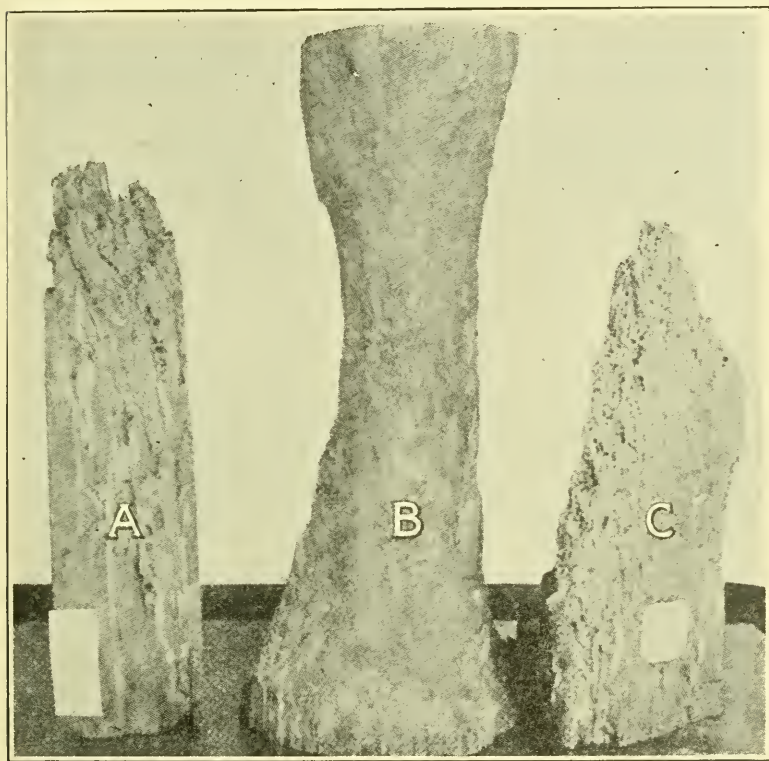


FIG. 9.—Results of limnoria and teredo action on piling. (*Trans. Am. Soc. C. E.* 1922, p. 1413.)

The limnoria and chelura are crustaceans, which attack wood on the outside for a limited distance above and below low-water level.

There are also some forms of fresh-water wood borers.

Figures 9, 10, and 10a show the work of the limnoria and teredo. These are taken from the discussion by W. G. Atwood, in the *Trans. A.S.C.E.* for Aug., 1922, pages 1413 and 1421.



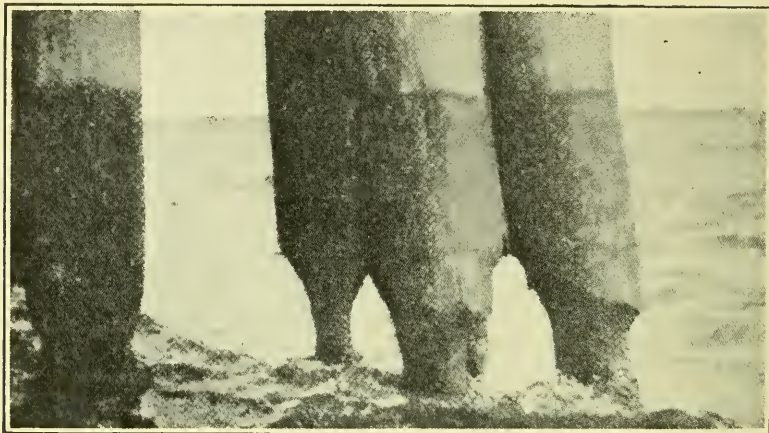


FIG. 10.—Limnoria attack on piles on Atlantic seaboard. (*Trans. Am. Soc. C. E.*, 1922, p. 1413.)

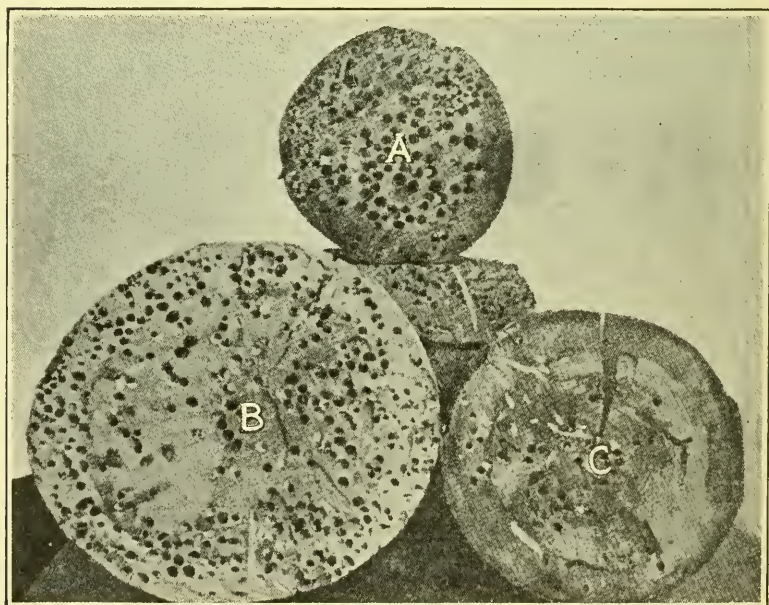


FIG. 10a.—Douglas fir piling after four years' service. (*Trans. Am. Soc. C. E.*, 1922, p. 1421.)

An excellent discussion of this subject, indeed, the best that the writer knows, is contained in the three reports of the San Francisco Bay Marine Piling Committee for 1921, 1922, and 1923.

**19. Preservation of Timber.**—There are several methods used to protect wood against decay; (1) Seasoning; (2) an outside coating; (3) interior preservatives.

Seasoning tends to preservation because it dries out the sap. It may, however, cause checking and so reduce the strength and provide a lodgment for germs, which all come from the outside. It is beneficial as far as decay is concerned, but seasoned woods that are to be exposed to the weather should be protected by external coatings or by internal antiseptics.

Outside protection by paint is generally practiced and excludes germs. A poisonous coating like creosote is often effective. Piles are sometimes protected by being covered with nails with large heads, by copper, or other metal, by cement or concrete, or other coatings. These may be effective as long as they remain, but they may be destroyed by blows or abrasion. Charring the outside is often effective. This method is used on fence posts.

Inside preservatives are applied either by soaking the timber, or by putting it in a vacuum to extract as much sap and moisture as possible and then soaking in the preservative, or by forcing the preservative in by pressure. Many preservatives have been tried, but the two now most used are creosote and zinc chloride, though corrosive sublimate (kyanizing) is sometimes used. Creosote is the most effective, and is the only one that is effective against wood borers. Creosoted piles have withstood the attacks of the teredo for as long as 40 years.<sup>1</sup> The great advantages of creosote are that it is not soluble and therefore will not wash out, that it is poisonous to germs and wood borers, and that it does not injure the wood. Its disadvantages are its odor, its black oily appearance, and the increased inflammability. Zinc chloride is the cheapest preservative, but is easily washed out and injures the timber. In dry locations it may be effective for many years. The use of zinc chloride is known as Burnettizing. There are standard specifications for these preservatives and for their use.

The quantity of creosote injected should depend on the use. Railroad ties sometimes have as little as 6 to 8 pounds per cubic foot injected, piles from 12 to 24 pounds per cubic foot.

<sup>1</sup> SNOW.



**20. Slow-burning Wood.**—Woods that have been treated with certain chemicals are sometimes termed “fire-proofed wood”; but wood cannot be made so that it will not burn. Salts which have water of crystallization, which is given off under heat, and some which give off non-inflammable gases, that retard combustion, may be injected into it but neither of these are of much efficiency. Others, on decomposing, leave behind a non-volatile fluid residue which covers the wood with a thin glaze which keeps out the air; of these, the most efficient is phosphate of ammonium.

There are paints which are not themselves inflammable, but they soon blister. Fireproof doors in factories are covered with tin, which protects the wood.

**21. References.**—With reference to decay, preservation, and wood borers, see Snow, who has especially good chapters on these subjects, with voluminous references.

Also Hoxie: “Dry Rot in Factory Timbers,” published by the Associated Factory Mutual Fire Insurance Cos., 184 High St., Boston.

Handy Book on Painting, published by John T. Lewis & Bros. Co., Lafayette Building, Philadelphia, Pa.

## CHAPTER III

### THE CONSTITUTION, HEAT TREATMENT, AND MECHANICAL TREATMENT OF IRON AND STEEL

1. The *constitution* of iron or steel is different from its *composition*. The latter term refers to the proportions of the different elements, while the former refers to the condition in which those elements or their combinations are found. Two steels may have the same composition, but very different constitutions and physical characteristics, depending upon the heat treatment or the mechanical treatment to which they have been exposed. The constitution is often called the "proximate composition," the composition, the "ultimate composition." The latter is the field of the chemist; the former is that of the metallographer. As Sauveur says "The analytical chemist may tell us, for instance, that a steel which he has analyzed contains 0.50 per cent of carbon, without our being able to form any idea as to its properties, for such steel may have a tenacity of some 75,000 lb. per square inch or of some 200,000 lbs., a ductility represented by an elongation of some 25 per cent or practically no ductility at all; it may be so hard that it cannot be filed or so soft as to be easily machined, etc." The structural engineer should be informed regarding these matters, the changes which take place in the solidification and cooling of iron and steel from a molten state, the effect of heat treatment and mechanical work, and the substances which are produced, as shown by the "constitutional" or "equilibrium" diagram. These matters are fully explained in the books mentioned in Chap. I, particularly clearly and concisely in those of Stoughton and Mills, and more in detail in those of Rosenhain and Upton. The following fundamentals should be familiar to the engineer.

2. In a liquid solution of one substance in another, the two, while not chemically combined, are so intimately mixed that they virtually form one liquid in which no indication of heterogeneity can be discovered by any physical instrument, even a

microscope or a polariscope. The solution may be in varying proportions, up to a maximum, or saturated solution, and the solution has properties which partake of the properties of the constituents. A chemical compound, on the other hand, contains the constituents in single definite proportions, it is a single definite substance, and it may, and generally does, have properties entirely different from those of either constituent.

If a liquid solution freezes, the constituents may or may not separate. If it freezes as a single substance, it is a *solid solution*, though not a chemical combination; and the constituents may change, as by precipitation, or change of crystalline form, even while solid, under variations of temperature or pressure. A liquid solution cannot freeze as a solid solution unless the constituents can crystallize alike. A metal always solidifies by forming crystals. If the different constituents crystallize in different forms, each would form its own crystals in freezing, and they would separate.

In a solid solution, as in a liquid solution, the dissolved substance may exist, in general, in various proportions up to a maximum or saturation point. This saturation point may be different in a solid solution from that of the liquid solution from which it is formed. If the saturated solid solution contains less of the dissolved substance than the liquid solution from which it is formed, the excess will separate out when solidification occurs.

**3.** Molten iron or steel is a liquid solution, mainly of carbon in iron. Pure iron, the element, does not occur apart from other elements, though crystals of nearly pure iron (ferrite) occur in all commercial irons and steels. The most important impurity is carbon, though there are others, such as silicon, phosphorus, sulphur, and manganese, which have important effects upon the properties of the resultant metal. For the present, and in the "constitutional" diagram, only the effect of carbon is considered.

Carbon dissolves in molten iron up to a maximum of about 7 per cent, though rarely over 4.5 or 5 per cent occurs. It forms a solid solution with iron up to about 1.7 per cent. Solid iron or steel may therefore have up to this proportion of carbon in solid solution, but no more; if there is more carbon in the metal, it is not in solid solution, but is separated out, in either one of two forms, namely, (1) *cementite* ( $\text{Fe}_3\text{C}$ ) which is a chemical com-

pound (not a solid solution) containing 6.67 per cent carbon,<sup>1</sup> or (2) *graphite*, which is nearly pure amorphous carbon.

4. In the diagram Fig. 11, ordinates are temperatures centigrade, and abscissas are percentage of carbon. It shows the changes that occur as the liquid solution cools. Above the lines *AE* and *EC* the metal is a true liquid solution, though authorities are divided as to whether it is a solution of carbon in iron or of a carbide of iron in iron. *AEC* is the line where metal of any composition will begin to solidify as it cools, and is called the *liquidus*,

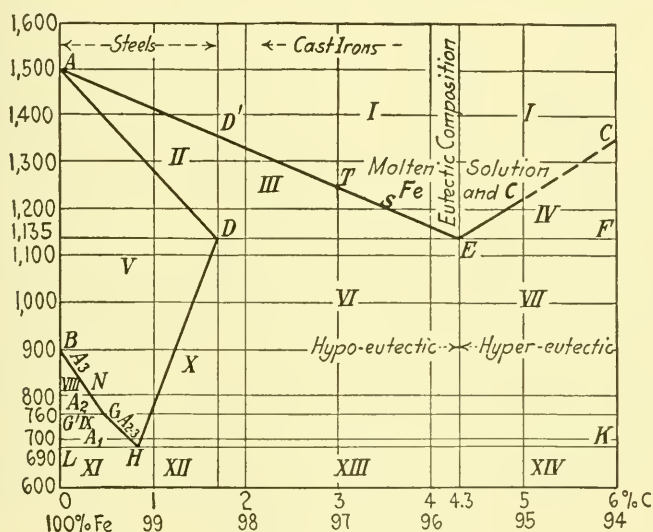


FIG. 11.—Equilibrium diagram of iron and steel.

or lower boundary of the region in which the metal is completely liquid. If the percentage of carbon is less than about 1.7, the metal will freeze as a solid solution of carbon in pure iron (or of iron carbide in iron). Thus, if there is 1 per cent *C* it will freeze as a solid solution of the same proportions. When it solidifies, the iron is in one of its three allotropic forms (see Art. 5), and is called  $\gamma$ -iron. The solid solution is called *austenite*, which may therefore have any proportion of *C* up to about 1.7 per cent, at which point it is saturated with carbon.

If the percentage of *C* is over 1.7 and less than about 4.3, there is too much *C* for the metal to freeze as a solid solution; hence

$$\frac{1}{3} \times \frac{12}{56 + 12} = \frac{1}{15} = 6.67 \text{ per cent.}$$

saturated austenite will separate when the temperature falls to the line  $AE$ , and the remaining solution will be enriched in carbon. The more  $C$  in the liquid, the lower its freezing temperature, and as the temperature falls and more and more saturated austenite separates, the composition of the remaining liquid follows down the line  $AE$ . If, for example, the original solution had 3 per cent  $C$ , it would begin to solidify at about  $1,250^{\circ}$ , at point  $T$ ; and as the temperature falls below this, saturated austenite separates. When the temperature has fallen to  $1,200^{\circ}$ , the liquid will have the composition indicated by the point  $S$ , or about 3.6 per cent  $C$ .

If there is over 4.3 per cent  $C$ , the first substance to separate as a solid is either graphite or cementite, according to circumstances, and this precipitation leaves the liquid with less  $C$  than before; and, as the temperature falls, its composition follows down the line  $CE$ , while graphite or cementite solidifies.<sup>1</sup>

The point  $E$  represents the composition of the liquid when it must solidify as a whole, forming an alloy (not a solid solution) of saturated austenite and either cementite or graphite. The solution at this point is called the *eutectic*, and represents the composition having the lowest melting point. A solution having the precise eutectic composition (4.3 per cent  $C$ ) would not solidify in cooling until the temperature fell to  $1,135^{\circ}$ , and would then immediately form saturated austenite and either graphite or cementite (or both). Below 4.3 per cent  $C$ , cast iron is *hypo-eutectic*; above 4.3 per cent  $C$  it is *hyper-eutectic*.

The line  $EC$  is slightly different according as graphite or cementite is precipitated from a solution having over about 4.3 per cent  $C$ . Slow cooling favors the formation of graphite, rapid cooling that of cementite. Similarly, the cementite which forms when the eutectic freezes may be later separated into ferrite and graphite, according to the rate of cooling and the other impurities present. Depending on the relative effects of the various factors, the metal may be a white cast iron, with almost all the carbon combined in cementite, or a gray cast iron, with the carbon in the form of graphite.

The substances in the different areas of the diagram are therefore the following:

<sup>1</sup> . . . or, cementite may first separate in every case, and at the high temperature existing may be decomposed into graphite and ferrite.



I: A liquid solution of carbon or carbide of iron in iron;

II: A mixture of solid austenite ( $C$  in  $\gamma$ -iron) in a liquid solution. The austenite is saturated (1.7 per cent  $C$ ) at  $DD'$ . Freezing begins on  $AE$  and ends on  $AD$ .

III: A mixture of solid saturated austenite in a liquid solution. Freezing ends at  $DE$  where the metal has the eutectic composition.

IV: A mixture of solid cementite or graphite in a liquid solution. Freezing ends on  $EF$ , where the metal has the eutectic composition.

The eutectic freezes on  $DF$ . The line  $ADEF$  is called the "solidus," and is the upper limit of the region in which the metal is completely solid.

V: A solid solution, austenite;

VI: A mixture of larger crystals of saturated austenite which crystallized in passing through III, and the eutectic, which solidifies as an intimate mixture of smaller crystals of saturated austenite with cementite or graphite. As the solid cools through VI, the austenite passes through changes still to be explained when we study the cooling of austenite below  $AD$ ; it becomes ferrite and cementite.

VII: A mixture of the cementite or graphite which solidified as the solution cooled through IV, with the solidified eutectic. A metal with exactly the eutectic composition would consist of cementite, graphite, or both, with saturated austenite.

The structure just below  $AD$  is a homogeneous solid solution, while below  $DF$  it is heterogeneous.

5. Pure iron (ferrite) exists in three allotropic forms:  $\alpha$ -iron, which is magnetic, and which is the form below the line  $G'GHK$ ;  $\beta$ -iron, which is non-magnetic, and occurs in area VIII; and  $\gamma$ -iron, which is non-magnetic, and which is the form in austenite.

Consider now the cooling of *solid steel* below the line  $AD$ . At  $AD$  it is white hot austenite, and as this solid solution cools it changes its constitution in a manner similar to that in which a liquid solution changes its constitution in passing through area II. If there is less than about 0.5 per cent  $C$ , the cooling metal will meet the line  $BG$ ; when it does, the austenite is immediately decomposed, even though solid, ferrite in the  $\beta$ -form separates, and therefore the remaining constituent becomes enriched in carbon, and follows down the line  $BG$  in composition. Thus, a



metal with 0.2 per cent  $C$  at  $800^{\circ}$ ,<sup>1</sup> would consist of  $\beta$ -ferrite together with austenite corresponding to the point  $N$ , that is, with about 0.35 per cent  $C$ . At the line  $G'G$ , the  $\beta$ -ferrite changes to  $\alpha$ -ferrite and becomes magnetic,<sup>2</sup> while with further cooling the austenite follows down the line  $GH$ . Thus  $\alpha$ -iron occurs up to a temperature of about  $760^{\circ}$  provided there is less than 0.5 per cent  $C$ , and if there is over 0.5 per cent  $C$  up to a lower temperature, varying with the percentage of carbon, and about  $690^{\circ}$  if the carbon is 0.85 per cent.  $\beta$ -iron only occurs if there is less than 0.5 per cent  $C$ , and at temperatures from  $760^{\circ}$  to a point varying with the percentage of  $C$  from  $760^{\circ}$  at 0.5 per cent to about  $900^{\circ}$  when there is no carbon.

If there is more than about 0.85 per cent carbon,<sup>3</sup> the cooling metal will meet the line  $HD$ , and where it does the austenite will be decomposed, cementite will be separated, the remaining austenite will be poorer in carbon  $C$  than before, and will follow down the line  $DH$ . Thus  $H$  is a point similar to  $E$ , except that it is in the solid. The composition at  $H$  is that of a solid eutectic, or *eutectoid*, as it is called to distinguish it from  $E$ .

Thus a steel with 0.2 per cent  $C$  in cooling from the liquid state would remain as austenite down to about  $840^{\circ}$  when  $\beta$ -iron would separate; at about  $760^{\circ}$  the  $\beta$ -iron changes to  $\alpha$ -iron, and at about  $700^{\circ}$  there is  $\alpha$ -iron and the eutectoid. A steel with 1.4 per cent  $C$  would remain as austenite down to about  $980^{\circ}$ , when cementite would separate, and at  $700^{\circ}$  there would be cementite and the eutectoid.

The eutectoid is called *pearlite*, and consists of a very finely divided mixture of minute crystals of  $\alpha$ -ferrite and cementite, into which the austenite separates at about  $700^{\circ}$ .

The remaining areas of the diagram therefore consist of:

VIII:  $\beta$ -ferrite and austenite;

IX:  $\alpha$ -ferrite and austenite;

X: "free cementite" and austenite;

XI: previously formed  $\alpha$ -ferrite, and pearlite;

<sup>1</sup> All temperatures are centigrade.

<sup>2</sup> Some metallurgists believe that the line  $GG'$  does not mark an allotropic change, and that  $\beta$ -iron does not exist. They think that  $GG'$  is merely the lower limit at which all the iron has been changed from  $\gamma$  to the  $\alpha$ -form.

<sup>3</sup> Some books give the eutectoid composition as 0.9 per cent  $C$ .

XII: previously formed cementite ("free cementite"), and pearlite;



FIG. 12.—Micrograph of Armco ingot iron, or ferrite, magnified 125 times.

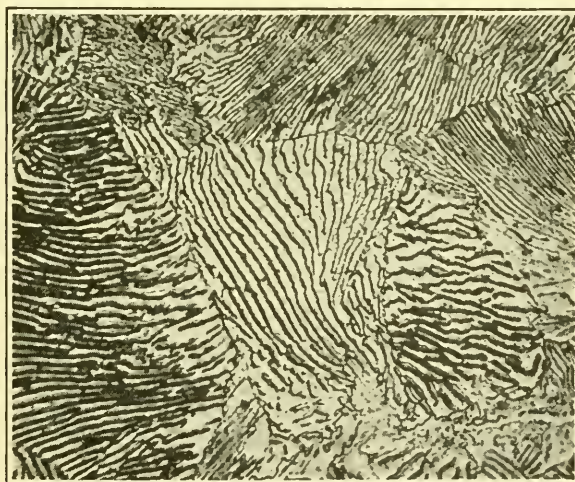


FIG. 13.—Micrograph of eutectoid steel (0.85 per cent C) showing finely laminated pearlite; magnified 300 times.

XIII and XIV: previously formed cementite or graphite, and pearlite.

In Fig. 12 is shown a micrograph of Armco Ingot iron, almost pure ferrite. Figure 13 shows eutectoid steel, or finely lami-

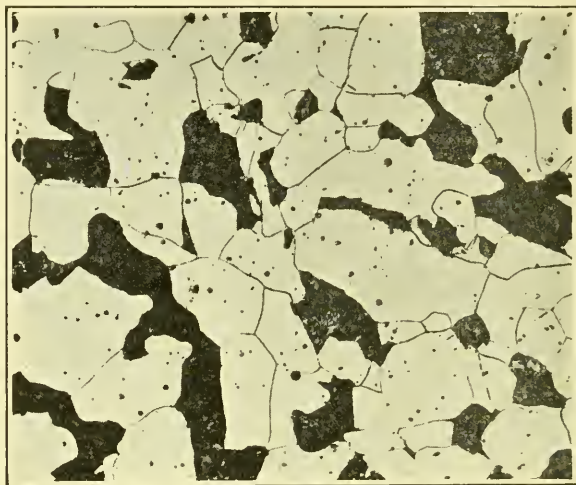


FIG. 14.—Hypo-eutectoid steel (0.35 per cent *C*); the dark is pearlite, the light ferrite; magnified 100 times.

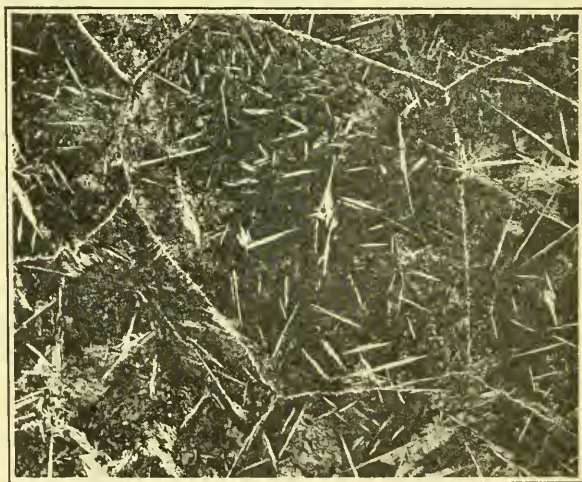


FIG. 15.—Hyper-eutectoid steel; the dark is pearlite, the light is cementite nated pearlite; Fig. 14 shows hypo-eutectoid steel (0.35 per cent *C*), showing pearlite and ferrite; Fig. 15 is hyper-eutectoid steel.

For all the micrographs in this book the writer is indebted to his colleague, Professor A. Sauveur.

6. Alloys with less than 1.7 per cent *C* are classed as steels; those with over 2.2 per cent *C* are cast irons; those with from 1.7 to 2.2 per cent are intermediate, and are not represented by commercial products. Graphite is not found in steels, except possibly in a very high-carbon steel verging on cast iron. Pure iron in the  $\gamma$ -form never occurs: it is always in solid solution with carbon as austenite. But if we could start with pure iron and heat it, it would successively be  $\alpha$ -iron,  $\beta$ -iron, and  $\gamma$ -iron.

A steel with carbon below about 0.2 per cent is called a low-carbon steel; with from 0.2 to 0.5 a medium steel; with from 0.5 to 0.6 hard steel, and above 0.6 a high-carbon steel; though these limits are not standardized.

Sauveur (*loc. cit.* 1912, p. iv, 1) says that the following terms are those most commonly used:

Very low carbon steel, very mild or extra mild steel, very soft or dead soft steel...	Carbon not over 0.10 per cent
Low carbon steel, mild steel, soft steel....	Carbon not over 0.25 per cent
Medium high carbon steel, half hard steel.	Carbon 0.26 to 0.60 per cent
High carbon steel, hard steel.....	Carbon over 0.60 per cent
Very high carbon steel, very hard or extra hard steel.....	Carbon over 1.25 per cent

7. It has been assumed thus far that carbon was the only impurity. There are others in all irons and steels, and in the "alloy steels" some constituents are added in considerable amounts to produce certain qualities. These change the various points of the diagram; silicon and phosphorus, for instance, having the effect of moving points *D* and *E* to the left. The lines of the diagram are determined by experiment, and some which have been drawn straight should, if accurate, be curved. There are other modifications which have not been shown.

Considering only binary alloys, or those of two elements only, it is evident that in cooling they may act in different ways if one dissolves in the other when liquid. There are two extreme cases:

(a) The two can crystallize in the same form, and in solidifying they crystallize together, each crystal being composed of the two elements just as the liquid solution, no matter what the proportions of the two. This is a solid solution.



(b) The two crystallize differently, and separate entirely in solidifying, the state of solution existing when liquid being entirely destroyed, and the solid being composed of separate crystals of the single constituents.

There are intermediate conditions, in which there may be a solid solution up to a certain proportion of one element in the other, and a separation when that proportion is exceeded, as in the case of iron and carbon, as above explained.

Solutions of more than two elements give rise to more complicated conditions, which are explained in the detailed works on the subject.

As a liquid alloy solidifies, the parts which freeze first push away the still liquid parts, and the former therefore contain an excess of the primary metal, while the parts which solidify last contain a larger proportion of the eutectic and other relatively fusible constituents.

8. We have seen that the solid solution, austenite, in cooling below the line  $AD$ , Fig. 11, undergoes molecular changes when the lines  $BGH$ ,  $G'G$ ,  $LH$ , and  $HD$  are crossed. These changes are accompanied by the evolution of heat, or retardation of the rate of cooling, shown by an increase in the time necessary to cool a given number of degrees; this occurs when the austenite is decomposed on the line  $BG$  with separation of  $\beta$ -ferrite, also on  $G'G$  when the  $\beta$ -ferrite changes to  $\alpha$ -ferrite; and a still greater evolution of heat, with glowing of the metal, known as recalescence, when the eutectoid becomes pearlite on  $LH$ . These change-points, beginning with the one at the lowest temperature, are known as  $A_1$ ,  $A_2$  and  $A_3$ . The points of corresponding change when heating are not at precisely the same temperatures as the points when cooling, owing to a lag or hysteresis which retards both transformations. The lag depends upon the rapidity of heating or cooling, being smaller the slower the temperature change, and upon other circumstances; so that the points of change on heating are called  $Ac_1$ ,  $Ac_2$ ,  $Ac_3$ , and those on cooling  $Ar_1$ ,  $Ar_2$ ,  $Ar_3$ ;  $c$  standing for *chauffage*, or heating, and  $r$  for *refroidissement*, or cooling. If the composition is to the right of  $G$  there are only two points, namely, where  $LH$  is crossed ( $Ac_1$  or  $Ar_1$ ) and where  $GH$  or  $HD$  are crossed, and the latter is known as  $Ac_{2-3}$  or  $Ar_{2-3}$ . If the composition is precisely the eutectoid, with about 0.85 per cent carbon, there is but one point, at  $H$ , known as  $Ac_{1-2-3}$  or  $Ar_{1-2-3}$ . The higher the carbon, in  $LH$ ,

the nearer the points 2 and 3, or the points 1 and 2-3. The temperature between the two corresponding points for heating and cooling, between  $Ac_3$  and  $Ar_3$  for example, may be from  $25^\circ$  to  $50^\circ$  C.

Temperatures from  $B$  (about  $895^\circ$ ) down to  $L$  (about  $690^\circ$ ) for steel having less than 0.85 per cent carbon (known as hypoeutectoid steel) and from  $D$  (about  $1,135^\circ$ ) down to  $H$  ( $690^\circ$ ) for steel with above 0.85 per cent carbon (known as hyper-eutectoid steel), constitute the *critical* or *transformation range*. Above this range the steel is a homogeneous solid solution, austenite; while below  $LHK$  it is called by Howe "a conglomerate or granitic mass, a mixture of pearlite with either ferrite or cementite according to whether it contains less or more than 0.85 per cent of carbon." For any given steel, the critical temperature is on the line  $BGHD$ , though, for reasons that will be referred to,  $HD$  is not often used, and the critical temperature is often considered to be on  $BGHK$ .

The properties of steel are extremely variable, depending not only upon the amount of carbon and other elements in it, but particularly upon the heat treatment and the mechanical treatment it has received. The engineer should be familiar with the facts and principles involved.

Ferrite, by itself, is soft and ductile; cementite is very hard and brittle, and it is not desirable to have much of it in steels for structures or machinery, where exposed to shocks or blows, or where a failure would be serious or involve loss of life. "Free" cementite, or that formed above the line  $DH$  is undesirable; that present should preferably be in pearlite, that is, very finely divided and mixed with ferrite, and not in large crystals. Even in tool steel, which is exposed to blows, the cementite should if possible be as pearlite. Below 0.85 per cent  $C$  there is more pearlite as the carbon increases; above 0.85 per cent  $C$  there is less pearlite as the carbon increases.

All the carbon in the steel is in the cementite, which contains  $\frac{12}{180} = \frac{1}{15}$  of its weight of carbon. The percentage of cementite in steel is therefore 15 times the percentage of carbon. Steel of the eutectoid composition contains about  $0.85 \times 15 = 12.75$  per cent of cementite and 87.25 per cent of ferrite. Almost all steels for structures or machines have less than 0.85 per cent  $C$ ; those with more, or to the right of  $H$  in Fig. 11 are tool steels and some spring steels.



If a hypo-eutectoid steel is heated from the conglomerate or granitic condition in which it exists at ordinary temperatures, at the line  $LH$ , or point  $Ac_1$ , the pearlite is immediately transformed into austenite. With further heating, the austenite progressively absorbs the ferrite with which it was originally mixed. (To the right of  $H$  it absorbs the cementite.) When the  $\alpha$ -iron changes to  $\beta$ - or  $\gamma$ -iron it loses its magnetism, and when it reaches area  $V$  it is all austenite solid solution, the carbon or carbide being entirely dissolved in  $\gamma$ -ferrite. Below  $Ac_3$  there are ferrite islets diffused in the austenite; these ferrite grains become coarser as the heating goes on, particularly in low-carbon steels, and this grain growth is accelerated by previous over-strain by cold working; but at  $Ac_3$  they are shattered and become very fine.

For reasons that will be explained presently, it is desirable that the grain of steel be as fine as possible. This will be accomplished if it is heated to just above the line  $BGHK$ . If that line is barely crossed, particularly in hypo-eutectoid steels, the grain will be the finest possible owing to the breaking up of the ferrite crystals at  $Ac_3$ . It must be kept at that heat only long enough to allow the complete diffusion of the last of the ferrite in the austenite. To heat higher would hasten this diffusion, but would coarsen the austenite grains by contact and coalescence. Long and high heating above  $Ac_3$ , followed by slow cooling without mechanical work would coarsen the grain and hinder the transformations in subsequent cooling, tending to preserve the coarsened grain of the austenite, which will be coarser the higher the heat and the smaller the percentage of carbon. Coarse grain is accompanied by reduction of tensile strength, the more as the carbon is less; and by reduction of ductility, the more as the carbon is greater. The finest grain will be in steel of the eutectoid composition, heated to just above  $H$ ; because above  $LH$  the new austenite grains grow, and yet the old grain size of the ferrite cannot be reduced till  $Ac_3$  is reached. It is the same with steels above 0.85 per cent carbon; the old grain size of the cementite cannot be reduced till  $HD$  is crossed, but this line is so steep that if heated to  $HD$  the austenite is coarsened, so that it is better to heat to just above  $HK$ , particularly since the excess or "free" cementite is small.

9. If steel which has been heated above the critical temperature is allowed to cool slowly without mechanical work, the grains

grow, more slowly as the temperature falls, depending upon the composition. If it has been heated far above  $A_{c3}$  it will be very coarse. If it has been heated nearly or quite to the melting point, it will be burned, and will contain oxides or gases: such steel is generally ruined, and can only be made good by remelting. If very low-carbon steel (under 0.15 per cent  $C$ ) is kept for several days at a temperature from  $500^{\circ}$  to  $750^{\circ}$ , or below  $A_{c1}$ , the crystals become very large, provided it has been cold-worked, and the strength and ductility are greatly reduced: this is known as "Stead's brittleness."

**10. Hardening.**—The molecular transformations described require time. If sufficient time is not allowed, that is, if the temperature changes too rapidly, there is the lag above referred to, and the changes take place at a higher temperature in heating and at a lower temperature when cooling. Further, the change of austenite into pearlite is not abrupt, but there are three intermediate stages recognized, namely, *martensite*, *troostite*, and *sorbite*. Martensite is the hardest, strongest, and least ductile, and the others are progressively less hard, less strong and more ductile, till pearlite is reached, which is the softest, weakest, and most ductile. (Cementite, however, a component of all steels, is harder than martensite.) If the cooling is sufficiently slow, the transformation is complete at about  $700^{\circ}$ , and the steel becomes pearlitic. If the cooling is very rapid, as by quenching in cold water, the transformation temperature may be lowered to about  $300^{\circ}$ , and the transformation is arrested at the stage of martensite. With intermediate rates of cooling, the temperature of transformation will not be reduced so much, and troostite or sorbite may be produced; and the properties of the steel will vary accordingly. In this way steel is hardened by sudden cooling from the critical temperature  $A_{c3}$  or above. Hardening is produced by any element or treatment which throws the cooled metal into the form of martensite or an intermediate product between austenite and a mixture of either ferrite and pearlite or cementite and pearlite.

We have seen that the finest grain will be produced if the steel is heated to just above the critical temperature and kept there just long enough for thorough diffusion of the ferrite (below 0.85 per cent  $C$ ). If the steel is suddenly cooled from this temperature, the fineness of grain will persist, and the steel will become fine-grained and hard; martensite, troostite, or sorbite being

produced according to the rapidity of cooling. If cooled from a higher temperature, the hardness will be no greater, but the grain-size will be larger. If heated ever so little below  $Ac_3$  and cooled, it will not be hardened. It should therefore be heated to just above  $Ac_3$  to produce the finest grain combined with hardness. The heating should be slow at the end, in order to bring all the metal to the desired temperature, and not heat the exterior far above it. In this hardened condition, however, it is so brittle as to be unsuitable for most uses; and, moreover, serious internal cooling stresses are produced, which must be removed.

If the steel is above 0.85 per cent  $C$ , the best temperature for hardening is just above the line  $HK$ , because that gives the best grain structure. If heated higher there would be more danger of cracking when quenched.

Hardening depends upon and increases with the percentage of carbon. Pure iron is hardened practically none at all by sudden cooling, and the same is true of the free ferrite which separates in steel below 0.85 per cent  $C$  and of the cementite which separates in steel above 0.85 per cent  $C$ . It is only the pearlite or eutectoid upon which hardening depends. Hardening therefore only affects the ferrite and cementite which forms the eutectoid, which on slow cooling becomes pearlite, but which on rapid cooling takes one of the intermediate forms. The  $\gamma$ -iron of austenite is harder than  $\beta$ - or  $\alpha$ -iron, and on rapid cooling some of it remains in the  $\gamma$  form. When it exists in this form at ordinary temperatures, in martensite, troostite, or sorbite, due to sudden or rapid cooling, it is not so stable as when produced by slow cooling, as it can be in some of the alloy steels; and these intermediate forms may therefore, when produced by sudden cooling of carbon steel, be more completely transformed toward pearlite by moderate heating.

All the carbon present in steel forms the carbide  $Fe_3C$ , which contains  $\frac{12 \times 100}{3 \times 56 + 12} = 6.67$  per cent  $C$ . Also, since 12 parts of carbon form 180 parts of  $Fe_3C$  or cementite, one part of carbon will form 15 parts of cementite. Hence the percentage of cementite is 15 times the percentage of carbon. Pearlite contains 0.85 per cent carbon, hence it contains  $15 \times 0.85 = 12.75$  per cent cementite, and therefore  $100 - 12.75 = 87.25$  per cent ferrite, or approximately 6.8 parts of ferrite by weight for each part of cementite. Again, one part of cementite in pearlite will

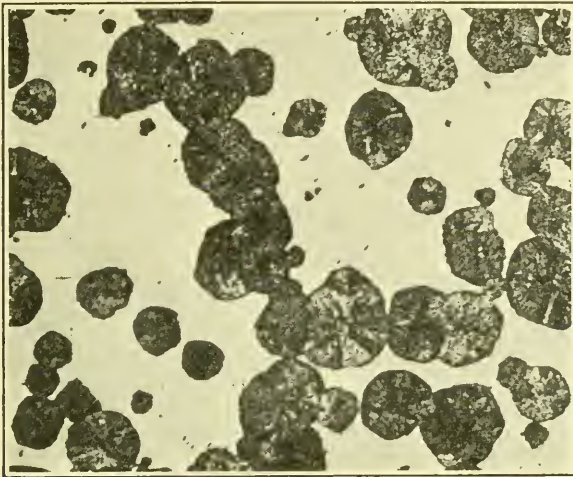


FIG. 16.—Hardened hyper-eutectoid steel; the light is martensite, the dark troostite.



FIG. 17.—Hardened hyper-eutectoid steel; the light is martensite, the dark troostite.



produce  $\frac{100}{12.75} = 7.8$  parts of pearlite; and one part of ferrite in cementite will produce  $\frac{100}{87.25} = 1.15$  parts of pearlite.

In *hypo-eutectoid steel*, all the carbon present, and hence all the cementite, is in the pearlite, hence hypo-eutectoid steel must contain a

$$\text{percentage of pearlite} = C \times 15 \times 7.8 = 117C$$

in which  $C$  = percentage of carbon.

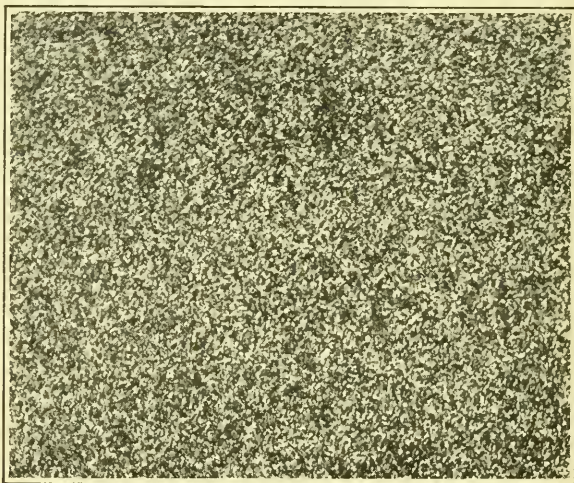


FIG. 18.—Sorbite in steel.

The balance is free ferrite.

In *hyper-eutectoid steel*, all the ferrite present is in the pearlite, and since

$$\begin{aligned}\text{percentage of cementite} &= 15C \\ \text{percentage of ferrite} &= 100 - 15C\end{aligned}$$

it follows that

$$\text{percentage of pearlite} = (100 - 15C)1.15 = 115 - 17.25C$$

The balance is free cementite.

Figure 16 shows hardened hyper-eutectoid steel, with martensite and troostite. Figure 17 shows a similar steel with smaller magnification, which brings out better the structure of the martensite.

Figure 18 shows a micrograph of sorbite; and Fig. 19 is martensite in hardened steel, magnified 800 diameters.

In order to produce homogeneity, it is desirable that the eutectoid and the free ferrite should be thoroughly intermingled; hence the desirability of heating to  $Ac_3$ , when the ferrite is thoroughly diffused in the austenite, and, when suddenly cooled, does not have time to separate and coalesce. This is one reason why a fine grain is desirable.

It is impossible to cool carbon steel fast enough to produce much austenite, though a little may be produced by sudden cooling of high-carbon steels; the most rapid cooling practicable produces martensite. In alloy steels, however, austenite may be

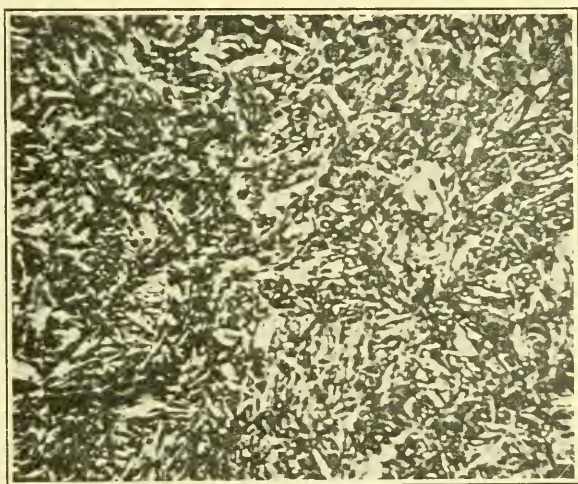


FIG. 19.—Martensite in hardened steel. Magnified 800 diameters.

produced. When pure iron is quenched, the ferrite crystals have merely to rearrange themselves in changing from the  $\gamma$  to the  $\beta$  and  $\alpha$  form, without changing their positions; but, as the carbon content increases, the austenite, in cooling, expels the freed ferrite into the inter-crystalline spaces, causing both it and the carbide to travel, and this requires time; hence the transformation is hindered if the cooling is sudden, and intermediate products are the result. Thus pure iron is not hardened by sudden cooling, and the effect is greater as the carbon increases. If quenched from above  $HD$ , martensite is produced, but it will be coarse; if the composition is that of the eutectoid, and the metal is quenched from just above  $H$ , the structure will be very



fine, and the product has been called *hardenite*, which is saturated martensite, or martensite with the eutectoid composition.

**11. Annealing and Tempering.**—Annealing is a softening treatment, consisting in heating a hardened steel to slightly above the critical temperature  $Ac_3$ , or to a lower temperature, and cooling slowly. If heated to just above  $Ac_3$ , the softest and most ductile material will be produced, the internal stresses most effectively removed, and the steel will be pearlitic; but the tensile strength and yield point will be lowered. The time for which the steel is held at the annealing temperature is important, a longer time at a lower temperature producing the same effect as a shorter time at a higher temperature. The time depends upon the thickness.

To give greater strength and yield point than would result from heating to  $Ac_3$ , while still retaining sufficient ductility without brittleness, the steel should be made sorbitic. This is done by heating to just above the critical point, thus refining the grain; then producing martensite by sudden cooling while preserving the fine grain, because in sudden cooling the ferrite or cementite grains do not have time to grow or coalesce as they may in slow cooling. Such hardened steel, however, is too hard and brittle for most uses, so that it is reheated to a temperature below  $700^\circ$ , kept there a suitable time, and then either slowly cooled or quenched, or quenched after cooling to a proper point to produce sorbite or whatever product is desired. This treatment is called *tempering* or *drawing*. Gradual separation of the finely divided pearlite, and coalescence of the cementite, which is undesirable, would be produced by long heating at  $700^\circ$ . In the second heating to some temperature below  $700^\circ$ , the metal is so rigid that coalescence of grains does not occur to a harmful extent, and by proper treatment sorbite, troostite, or an intermediate structure may be produced. This is therefore a *strengthening treatment*, which preserves the fineness of grain.

If great hardness is desired, and brittleness is not objectionable, the steel may be made martensitic by sudden cooling, or as much of the hardness as is objectionable may be removed by a slight tempering.

It would be possible to produce sorbite by a single cooling, properly regulated, or quenched at the desired point, but the grain would not then be as fine as by the tempering process.

Tempering is sometimes called “drawing,” or “drawing the temper.” In doing this, the desired temperature is estimated

by the so-called "temper colors" shown by the steel when heated, caused by the formation of a thin film of oxide on the surface. These colors are:

Pale yellow.....	220° C. = 428° F.	Brown-purple...	265° C. = 509° F.
Straw.....	230° C. = 446° F.	Purple.....	280° C. = 536° F.
Light brown.....	243° C. = 469° F.	Bright blue.....	288° C. = 550° F.
Brown.....	255° C. = 491° F.	Pale blue.....	300° C. = 572° F.
		Dark blue.....	315° C. = 599° F.

The higher the temperature of tempering, and the longer the steel is kept at that temperature, the softer will be the metal.

Steel which has been hardened by quenching is not much affected by subsequent heating to 100° C.; the particles are not mobile enough: but steel which has been stressed above the yield point, so that the elastic limit has been reduced to nearly zero, owing to the plasticity of the intercrystalline amorphous cement, rubbed off by the slipping, will have its elasticity entirely restored by a period of rest, which gives the amorphous material time to harden, or as Muir has shown,<sup>1</sup> by a short exposure to a temperature of 100° C.

By "heat treatment" is therefore to be understood hardening, tempering, annealing, normalizing, and similar operations.

*Normalizing* is annealing by heating to above the critical temperature for a certain time, and cooling in air, which is said to give a fine-grained, pearlitic structure. Normalizing, however, does not always leave the steel in the same condition. Much depends upon the size of the piece. A small piece would cool more rapidly than a large piece, and would become sorbitic, while a large piece would become pearlitic, or would have a different structure on the outside and on the inside. The strength and other properties are therefore, even in the same kind of steel, not necessarily made uniform by normalizing, though they may be made more uniform than they previously were.

Sudden cooling from above the critical range always produces internal stresses, especially in thick pieces, and is liable to cause cracks, especially if there were previous initial stresses. Hence a softening treatment is often given before hardening. A piece may be so thick that it cannot be hardened throughout, but only on the surface. A temperature  $A_3$  is too high for quenching any but small pieces. Cracks produced by quenching are often intercrystalline, as the cooling takes place suddenly when the

<sup>1</sup> *Phil. Trans. Royal Soc.*, 1899.

amorphous material is plastic. Cooling should only be as rapid as is consistent with avoiding undue cooling stresses; it should be slower the greater the least thickness of the piece.

If a low-carbon steel already has a fine grain, produced by mechanical working (see Art. 12), it may be made soft, and the internal stresses removed, by suitable heating at a temperature below the critical range, say at  $600^{\circ}$  C. Important castings, especially if thick, should be annealed to remove cooling stresses; and structural steel which has been heated subsequent to rolling, such as eye-bar heads, or which has been stressed by cold working above the elastic limit, should be annealed.

Iron in the  $\gamma$ -form and in the  $\beta$ -form is non-magnetic; only in the  $\alpha$ -form is it magnetic. The proper heat for annealing may be found by means of this property. For steel between  $G$  and  $H$  in composition the critical temperature is reached, in heating, when it becomes non-magnetic, since the iron is in the  $\alpha$ -form below, and in the  $\gamma$ -form above that line. If a magnet is suspended outside the heating furnace, the proper heat is reached when the iron no longer attracts the magnet. For steel between  $G$  and  $B$  in composition, since  $\beta$ -iron is non-magnetic, the steel becomes non-magnetic above  $GG'$ , while the critical temperature is on  $BG$ ; the difference can be estimated in heating.

**12. Effect of Mechanical Work.**—If steel is heated well above the critical point  $Ac_3$ , and allowed to cool slowly, we have seen that the grain will be coarse. This is only true, however, if no mechanical work is put upon it; if it is rolled or hammered, the crystals are broken down, by crushing or shearing, into smaller ones, and the final size of grain is *dependent upon the temperature at the finish of the mechanical working*. The greater the amount of mechanical work the better, and the working should be finished about at the line  $BGHD$ . There is here a directional effect, however; the more working there is in a given direction, as by rolling, the more the ductility in a transverse direction is decreased. Each passage through the rolls reduces the grain-size, and in the interval between passes it grows again. Work done on the metal below that line increases the brittleness, while if the work is finished at a higher temperature the strength will be less. Heavy working should stop somewhat above the line  $BGHD$ , moderately heavy working may be continued to lower temperatures, and light working still lower; the proper temperatures depending on the carbon content, and also on the percentage

of other impurities. As working below that line increases brittleness, great care should be taken that the steel is not worked too cold. It is very dangerous to do even light working on steel at temperatures between 150 and 550° C., especially between about 250 and 350°, which is the region of so-called blue heat. The temperature at which colors appear, however, depends somewhat on rapidity of heating.

The above shows the necessity of having a proper proportion between the size of the ingot and the size of the finished piece. If the ingot is too small, the finishing temperature may be too high; if the ingot is too large, the finishing temperature may be too low, unless the piece is reheated.

We thus see that if steel is heated considerably above the critical temperature  $Ac_3$  the grain may be made fine either by mechanical working or by heat treatment. It should never be heated above  $Ac_3$  without being refined in one of these ways.

**13. Initial Stresses Due to Mechanical Work.**—If steel is worked hot, initial stresses are not produced if the finishing temperature is high enough, except that they may be produced by unequal cooling; the working does not produce them. If it is worked cold, or below a temperature at which the crystals are mobile enough to adjust themselves, initial stresses are produced by the working, which are greater the lower the temperature at which the metal has been worked. Nevertheless, it is desirable to continue the working to below the temperature at which the grain coarsens by further cooling, for the coarseness is worse than the initial stresses and the so-called "strain-hardening."

It is generally stated that hot working does not harden the metal, while cold working, which involves stressing above the elastic limit, does harden, producing *strain-hardness*. This depends upon what is meant by hardness, and the different meanings of that word have been referred to in Chap. IV of the "Strength of Materials." Cold working, or stressing above the yield point, does of course harden in the sense that it raises the elastic limit and yield point, and sometimes the ultimate tenacity, diminishes the ductility or total stretch before fracture, and also reduces the ultimate resilience or capacity to resist shock; and increases the Brinell hardness. It makes the stress-strain diagram assume a shape like the line  $d_2dub$  in Fig. 59 "Strength of Materials," the origin being at  $d_2$ . Cold-worked metal has therefore, no doubt, less resistance to shock, greater brittleness (or less total

stretch), higher initial stresses, and probably, if the overstress has been severe enough, less resistance to repeated or alternating stresses.

Cold working may be by wire-drawing, rolling, or hammering.

*Cold-rolling.*—Let  $AC$  and  $HG$ , Fig. 20, be the surfaces gripped by the rolls as the piece is pushed through toward the right.

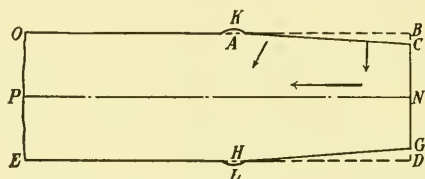


FIG. 20.<sup>1</sup>—The backward flow of metal during reduction is greater along the axis than on the surface of the bar and approximately in the direction shown by the arrows.

Disregarding the slight swelling at  $K$  and  $L$ , just back of the rolls, what has become of the original volume represented by  $ABC$  and  $HDG$ ? According to Howe and Groesbeck (A.S.T.M., 1920, Pt. II), the friction of the rolls prevents backward flow of the metal along the surfaces, but the inward pressure causes the metal in the central part of the piece to flow backward or in the direc-

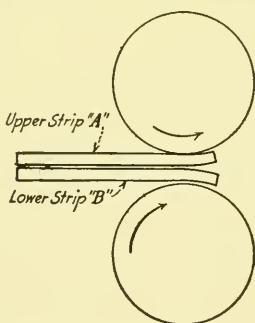


FIG. 21.<sup>1</sup>—Strip entering the initial pass.

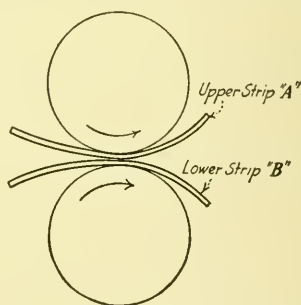


FIG. 22.<sup>1</sup>—A later pass.

tion of the horizontal arrow, producing initial longitudinal compression at the center, to balance which there must be initial longitudinal tension at the surface. To test this, two strips of equal thickness, Fig. 21, were put through the rolls together; they curved apart as shown in Fig. 22, convex to each other. This

<sup>1</sup> "A Note on Stresses Caused by Cold Rolling," by H. M. Howe and E. C. Groesbeck, (Am. Soc. for Testing Materials, Vol. XX, Part II, pp. 32 and 36, 1920.)



shows that in the solid piece the central part would be in longitudinal compression (since it would stretch longitudinally if split in the middle) and the surface in longitudinal tension, or else that there is transverse tension on a plane like  $PN$ , or both. It is difficult to conceive that transverse compression on the upper and lower surfaces can leave tension on  $PN$  (see below). R. W. Woodward etched away the upper half of a rolled piece, and the lower half curved like the lower plate in Fig. 22. We may therefore accept it as proved that there is a longitudinal compression at the center and tension at the rolled surfaces, on a plane perpendicular to the direction of rolling.

On a longitudinal plane, parallel to the direction of rolling and perpendicular to the piece, such as  $OEDB$  in Fig. 20, there should be little or no initial stresses if lateral expansion in rolling is prevented, as in rails or shapes, or plates rolled in a universal mill, which rolls all four edges. If lateral expansion occurs, the central parts are forced out laterally, and are left in compression, while the rolled surfaces are in tension.

On a longitudinal plane such as  $PN$  in Fig. 20, there is no reason why, with uniform rolling, the internal stress should vary in different parts, still less that it should be tension at some points and compression at others, as would have to be the case for equilibrium; hence there is no initial stress on such a plane, though it looks, at first sight, as though these would be the very planes on which there would be.

In *wire drawing* the wire is pulled through the die instead of being gripped between rolls. Here there may be backward surface flow rather than inward flow, leaving initial compression near the surface and tension near the center. Moreover, the surface is stressed higher than the inside, and more *strain-hardened* and strengthened. Sometimes wires draw hollow, or with breaks or cavities along the axis; or fail first at the center under a tension test.

*Hammering* is local, and tends to cause central flow in all directions, leaving compression at the center and tension at the surface.

It is often stated in discussions on cold working that, in drawing, tension remains at the surface and compression inside, while rolling and forging leave compression at the surface or just the opposite of the explanation above given.

Hot working does not produce these effects, but if the working



is continued to too low a temperature, the effect may be that of cold working.

The material close to sheared edges or punched holes has had the elastic limit raised by overstrain, and is often termed brittle, having less shock resistance and less ductility.

It is generally better to avoid the use of metal which has been severely cold-worked, in places where it may be subjected to shock or to alternating stresses, or else to remove the effects of the cold working by annealing.

On the other hand, it is desirable to use metal which has been properly hot-worked, and finished at a suitable temperature. Insufficient working means superficial working, that is, a fine-grained exterior and a coarse-grained interior. The proper finishing temperature may be ensured by providing a limit of shrinkage from the last pass through the rolls down to normal temperature. Thus the specifications of the A.R.E.A. and of the A.S.T.M. provide, in the case of steel rails, as follows:

The number of passes and speed of the train shall be so regulated that on leaving the rolls at the final pass the temperature of the rail will not exceed that which requires a shrinkage allowance at the hot saws, for a rail 33 feet in length, and of 100 pounds section, of  $6\frac{3}{4}$  inches, and  $\frac{1}{8}$  inch less for each 10 pound decrease in section (or  $\frac{1}{8}$  inch more for each 10 pound increase in section).

It appears, however, that the above shrinkage allowance is too great, and allows finishing some  $270^{\circ}$  C. above the critical range. In *Technologic Paper* 38 of the Bureau of Standards, it is even stated that this shrinkage clause "has no significance whatever."<sup>1</sup> However, stopping mechanical work at a temperature above the critical point will not give as large grain as merely heating to that point and slowly cooling; the effect of mechanical work persists. The manufacturer naturally likes to finish at as high a temperature as possible, as the rolling is easier.

There is also a difference between Bessemer and open-hearth steel as regards shrinkage, the above-named paper giving for Bessemer, coefficient of expansion = 0.0000146 per degree Centigrade; for open hearth, coefficient of expansion = 0.0000156 per degree Centigrade. There is a small contraction, in heating, at about  $700^{\circ}$ .

<sup>1</sup> See also *Trans. Am. Inst. Mining Eng.*, 1914, on "Finishing Temperatures and Properties of Rails."

**14. The So-called Cold Crystallization of Steel.**—Steel sometimes breaks in service—as a member of a bridge, a car axle, or other piece—and the break shows a crystalline fracture. It has often been claimed that this has been due to vibration or to repetition of stress, which has *crystallized* the metal. It is believed by the best authorities now that there is no such thing as *cold crystallization*. The crystalline appearance is due either to the manner of fracture, or to improper heat treatment. A piece of steel may be broken by a sudden blow, and show a crystalline fracture, because the parts did not have time to draw down and the break is across the crystals, without elongation; the same steel broken in a testing machine would show the usual ductility and reduction of area, with a silky fracture. It may also show a crystalline fracture, with no drawing-down, if such drawing-down is prevented by the fact that the piece was a notched specimen (see Chap. IV). As a matter of fact, since steel is composed of crystals, all fractures show crystals under the microscope; by *crystalline fracture* is therefore properly meant a coarsely crystalline structure. This can only be due to the heat treatment, or because the break is by fatigue (see Chap. XXII of the previous volume). There is no proof that vibration has caused, or can cause, large grain size in steel. If such grain exists, it existed before the steel was put into service, if it has not been heated, though the fact will not be disclosed till a fracture occurs.

**15. Welding of Steel or Iron.**—Parts of steel or iron may be welded together by being heated to a temperature near the melting point, when the material is soft and pasty, and pressing and hammering the parts together. The iron oxide which forms at such high temperature is dissolved by a flux. The parts to be welded are pressed together in such a way as to squeeze the slag out. The grain will be very coarse, owing to the high heat, if allowed to cool without working, so that the metal must be hammered continuously till below the line *BGHK*. The danger in welding is that some parts of the pieces, not at the weld, but a little away from it, will be heated to the area *V* without receiving the necessary work while cooling. A properly made weld will not break *at* the weld, but will often or generally break *near* the weld at a stress much less than the original strength, sometimes not over half. The defect to be feared is therefore coarse crystallization near the weld. To make a welded bar of proper strength, it should be reheated to the line *BGHK* to restore the grain. The

lower the carbon, the more easily steel will weld. It is generally considered that cast iron will not weld, and high carbon steel with difficulty; but by actually melting the adjoining parts, cast iron may be welded.

### EFFECT OF STRESS AND STRAIN

16. The engineer should know something of the effect of stress and strain on steel from the point of view of its crystalline composition. The following summary may assist him:

When the crystals form in cooling, they grow from a starting point or nucleus by coalescence or building up, as frost grows on a window pane. When fully formed, there will be interstices between the crystals, which will not fit accurately together. These interstices are not voids, but are filled, according to the best working hypothesis, by an amorphous or colloidal material formed during the original cooling, which has not yet crystallized,

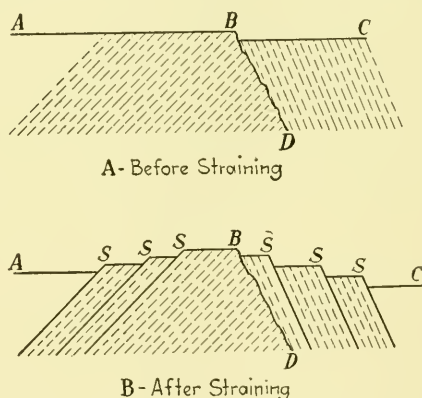


FIG. 23.—Diagram of the formation of slip bands. (Rosenhain).

perhaps because it is held in equilibrium between conflicting tendencies to crystallize with adjacent crystals. This amorphous material is an inter-crystalline cement, and, when hard, is stronger than the crystals themselves, as shown by the fact that a fracture is not around the crystals following the boundary lines, but through the crystals themselves. The boundary lines are surfaces of strength, not of weakness. At high temperatures, however, near the critical point, the amorphous material becomes weak and plastic before the crystals do, and then a tensile break will occur around the crystals and not through them, and there are few or no *slip bands* (see below).

Under tension, the steel, composed of a mixture of crystals of ferrite and pearlite, that is, of ferrite and cementite, with intercrystalline amorphous cement, will elongate, the ductile ferrite crystals elongating most, but being restrained by the intermingled brittle cementite, which elongate little. If the elastic limit is not exceeded, they resume their original shapes when the stress is removed. When the stress becomes great enough, there is slipping on oblique planes through the crystals, which would form a series of steps on a polished surface, as in Fig. 23, which would show as a series of parallel lines, or *slip bands*, on the surface of any one crystal, as the planes of slip would be parallel in that crystal. The planes of first slip would be the weakest planes. These planes, however, would have different orientation in adjoining crystals, and would not form continuous planes through the entire piece, but would be interrupted at the crystal boundaries, and therefore would show under the microscope as parallel lines on each crystal, but in different directions on the different crystals. This shows another reason why a fine grain increases the strength and elastic limit, because the slip planes are more often interrupted and changed in direction than if the grain is coarse. Another reason is that the finely divided brittle cementite crystals support better the intermingled ferrite crystals.

The slip bands, being steps, show dark under the microscope under normal illumination, because the light rays are dispersed by them and do not return into the microscope; but under oblique illumination, they may show white on a dark ground.

Figure 24 shows slip bands in ferrite. Figure 25 shows Neumann bands, which are only produced when the fracture is sudden, as explained in Chap. IV of the "Strength of Materials."

When slipping occurs, there is an abrasion or rubbing off of some material on the slipping surface, which thus becomes an amorphous layer along that surface. As long as this is plastic it acts like a lubricant, and the slipping continues, but it hardens quickly and regains its elasticity after a period of rest or under a moderate heat, say that of boiling water, and when it hardens, that slipping surface is stronger than it was before; so that a repetition of the same stress will cause no more slipping on those same planes, but slip will first occur on other planes, which are stronger, because there was no slip on them under the first load.



Thus there may be two non-parallel sets of parallel slip bands on the surface of the same crystal as seen in the microscope.

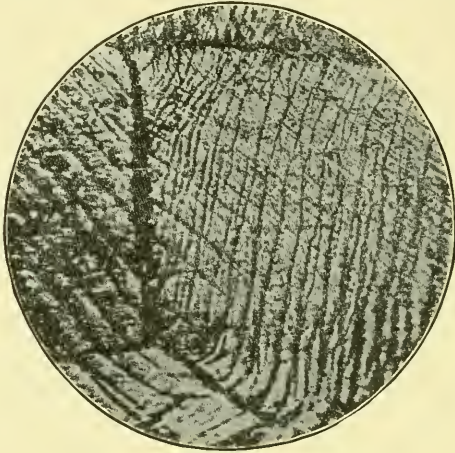


FIG. 24.—Slip bands in ferrite.

This theory explains fully the phenomena of the yield point, the immediate reduction of the elastic limit to zero by overstress,

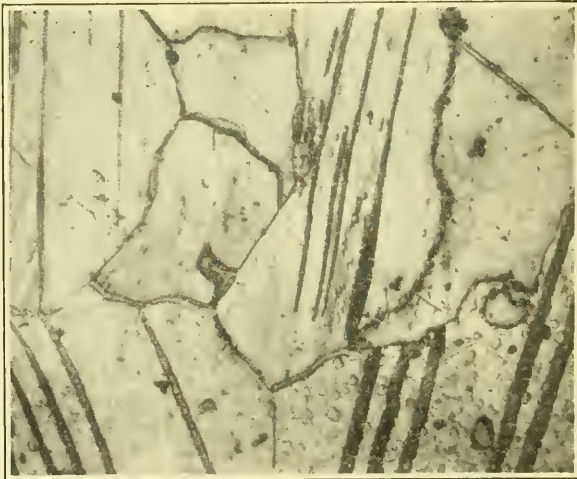


FIG. 25.—Neumann bands in ferrite. Magnified 125 diameters.

if retested immediately, the restoration of elasticity by a period of rest or by moderate heat, and the raising of the elastic limit

and ultimate strength by a stress above the yield point; but it does not explain the fact that an overstrain in tension lowers the elastic limit in compression and *vice versa*. Ultimately, under severe stress, when slip has occurred on many planes, the material becomes a mass of small crystalline fragments in relatively thick layers of amorphous material, and the hardness is generally increased while the ductility is diminished (because much of the final deformation has already occurred).

The character of a fracture will be different according as it is caused by a gradually applied load, by shock, or by repeated or alternating stress.

If caused by a gradually applied tension it will occur across the crystals, often along slip planes at some part of the surface. Since the crystals may be very minute, a fracture may look like a square break, and yet may be largely along shear planes. The fracture will look fibrous, especially if largely by shear.

A blow does not give time for slip, and the fracture is across the crystals on cleavage planes, which are not necessarily the same as the slip planes. The fracture may look crystalline.

Both fractures are across the crystals, not around them.

A fracture by repeated stress occurs by the formation of a minute crack along a slip plane, which gradually extends across the section. It is not easy to see how this crack could form unless at each repetition there was a slight increase of slip (deformation) along that plane. This lends probability to the maximum strain theory. The fracture often looks crystalline, because when the crack has progressed some distance the load on the remaining section becomes eccentric, and the break is sudden, as by a blow, and may then occur across cleavage surfaces. The method of development of a crack is another reason why a fine grain is desirable, especially in a piece which is to be subjected to alternating stresses or to shock; because a crack can be less easily propagated across finely divided crystals than across the slip surfaces of a smaller number of larger crystals. There is more of the strong amorphous material when the grain is fine, and a slip plane must change its direction at each crystal boundary, so that the yield point is increased by fineness of grain. Large crystals mean continuous slip or cleavage surfaces over larger areas.

In some alloys there may be brittle films of a different constituent surrounding the crystals, and the break may be through this constituent.



Under a gradual pull, the pearlite is apt to break before the more ductile free ferrite, because of the brittle cementite in the pearlite. The fracture, however, passes through both ferrite and cementite, though the strength of the latter alone is several times that of the former. Under alternate stress the fracture is more apt to originate in the ferrite, because it is weaker and slip bands form in it first.

Iron or steel broken slowly at ordinary temperatures show slip bands, but if broken slowly at a high temperature when the intercrystalline amorphous cement is viscous, the break will be around the crystals and there will be few or no slip bands. If broken suddenly at a high temperature, the viscous material has not time to flow, the fracture will be through the crystals, there will be slip bands, and the strength will be higher than if broken slowly.

17. The attempt has been made to give in this chapter a brief statement of the fundamentals which the engineer should know. The subject is a large and complicated one, and it is hoped that it has been made clear that the treatment of steel is a delicate matter, which affects the physical properties profoundly, and which should be under the supervision of a trained metallurgist.

It is hoped that the reader may be stimulated to continue his study of the subject. For this purpose he is advised to begin by reading Stoughton's book, and to follow it with Rosenhain's "Introduction to Physical Metallurgy." He should then study one, or better, all three of the general books on materials (Johnson, Mills, and Upton), and the paper by H. M. Howe, on "Life History of Network and Ferrite Grains in Carbon Steel," in the *Trans. A.S.T.M.* for 1911, pp. 262-386. Finally, he should study the large works of Howe and Sauveur, namely,

HOWE, H. M.: "The Metallography of Steel and Cast Iron," McGraw-Hill Book Co., Inc., 1916;

SAUVEUR, A.: "The Metallography and Heat Treatment of Iron and Steel," 1916.

## CHAPTER IV

### CAST IRON

1. Pig iron is the first product in obtaining iron or steel from the ore, and is obtained from the blast furnace. When pig iron is remelted and cast into molds, sometimes after mixing with other pig irons, it is cast iron.

The blast furnace, fully described in the books on materials, is charged with fuel (coke), ore, and flux (usually limestone) in alternate layers, and the iron oxide is reduced, trickles down to the hearth, and is drawn off at intervals. The flux unites with the ash of the fuel and the impurities or *gangue* of the ore, to form slag or cinder, which floats on the iron, and is drawn off separately, to be wasted or used in making cement. The character of the flux should therefore depend on that of the impurities. If these are acid, such as silica and alumina, as they usually are, the flux is basic, such as lime or magnesia, and the slag will consist mainly of silicates of lime or magnesia. If the impurities are basic, or high in lime or magnesia, the flux is acid. Most of the sulphur in the ore or fuel unites with the lime in the flux. If the impurities are both acid and basic in proper proportion, the ore is called self-fluxing. The character of the flux may be computed if the ore and fuel are known, and may be regulated so as to produce a desired result. The iron is cast into so-called pigs, or taken in ladle cars to the mixer, to be directly converted into steel or wrought iron or used for castings, without remelting, or more frequently remelted and mixed.

2. Cast iron is a complex material, and very subject to variation due to the character of the impurities and the details of operation of the furnace. It is coarsely crystalline, may be very hard, has a lower melting point than any other form of iron, but lacks toughness, and is not malleable or ductile, and cannot be welded except by smelting. It contains generally 3 to 4 per cent carbon (though sometimes up to 5 per cent or more), with varying amounts of other impurities, such as silicon, phosphorus, sulphur, and manganese, which greatly affect its structure and

properties. A typical cast iron would contain about 94 per cent iron, 3.5 per cent carbon, and 2.5 per cent other elements. It is of two principal grades, depending upon the condition in which the carbon exists in it, namely, *white cast iron* and *gray cast iron*.<sup>1</sup>

3. In *white cast iron* (Fig. 26) the carbon is mainly or entirely in the form of *cementite*, that is, chemically combined with the iron (ferrite). As this cementite ( $\text{Fe}_3\text{C}$ ) contains, by weight, one fifteenth carbon ( $\frac{12}{3 \times 56 + 12}$ ), it follows that with 3 or 4

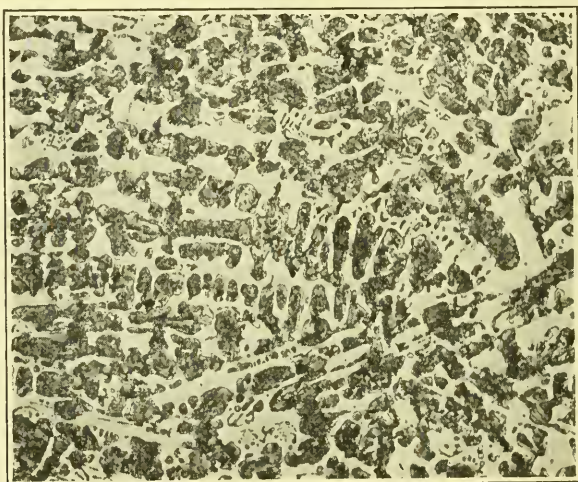


FIG. 26.—White cast iron: the dark is pearlite, the light cementite.

per cent carbon, if all combined, the cementite will constitute 45 or 60 per cent of the whole, by weight, the balance being ferrite and impurities. The properties of white cast iron will therefore be largely those of cementite; it will be hard, brittle, difficult to machine, and very resistant to abrasion. It has few uses, and none in structures or machinery, but is useful as the wearing surface of parts exposed to wear, as car wheels and brake shoes. It is produced by suddenly chilling the molten metal, as by casting it against an iron mold. The sudden cooling of the surface does not give the graphite time to separate as it does when

<sup>1</sup> The constitution and structure of iron and steel, and many important characteristics, cannot be understood without a good knowledge of the elements of metallography, a brief statement regarding which has been given in the preceding chapter, which it is hoped may lead to a study of the subject in detail.

the cooling is slow. Certain impurities, such as sulphur and phosphorus, favor the production of white cast iron, while silicon favors the separation of the carbon as graphite. The fracture of white cast iron is silvery white, with little or no graphite flakes visible, because they do not exist in this material. One of the main uses of white cast iron is for the production of malleable iron (see Chap. VI).

4. In *gray cast iron* (Fig. 27) the carbon is mainly in the form of graphite, deposited as flakes. Since iron is 3.5 times as heavy as graphite, it follows that if there were 4 per cent of carbon, all

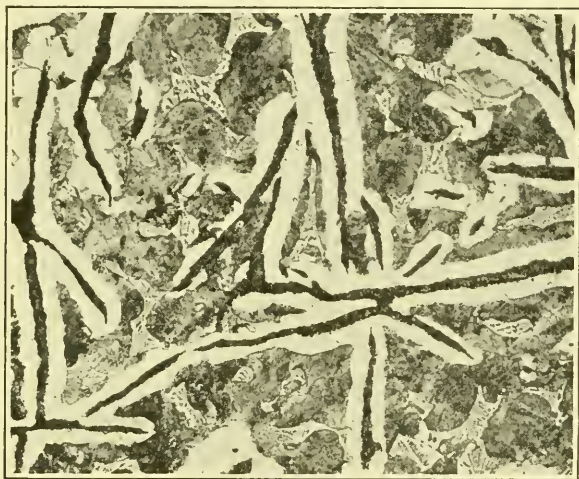


FIG. 27.—Gray cast iron; showing the black graphite; the ferrite as white borders around the graphite; the dark pearlite; and the phosphide eutectic (steadite).

in the form of graphite,  $1\frac{4}{10}$ , or about 12.5 per cent of the volume, and  $\frac{1}{4}$  of a plane surface, would be graphite.<sup>1</sup> Hence a fractured section would show the gray color of graphite, with perhaps little metallic luster unless the graphite particles were brushed off with a stiff brush. The actual percentage of graphite in gray cast iron, by weight, is from 2 to 4, the cementite being generally under 1.5 per cent. Graphite is very weak, and the particles easily split, hence gray cast iron is weak in tension or under shock.

Mottled cast iron is intermediate between white and gray. It has no uses.

$$^1 96 + 3.5 \times 4 = 110; 12.5 \text{ per cent} = \frac{1}{8}; \frac{1}{8} \times \frac{25}{8} = \frac{1}{4}.$$



**5. Uses of Cast Iron.**—Only gray cast iron is used in structures. It is not often used in tension. Formerly it was much used for beams, but is not now considered reliable for such use, and is only used for minor parts which are mainly or entirely in compression, as for bed plates, column bases, and columns in buildings.

On account of its cheapness, fluidity, high compressive strength, and the ease with which it may be cast, it is still one of the most useful engineering materials. About one-fifth of the pig iron produced in the United States is remelted and used for cast iron. It is used for car wheels, brake shoes, fly wheels, stoves, gears, water pipes, valves, and many machinery parts.

It appears from Bauschinger's tests that cast iron resists high temperatures, such as may occur in a conflagration, better than wrought iron; for although cast iron has a lower melting point, the temperature in a conflagration is generally far below such point, and at such temperatures as are reached cast iron bends less easily than wrought iron or structural steel. Both materials, however, if used in buildings, should be protected by fireproofing if damage by fire is to be avoided.

**6. Composition and Constitution.**—Gray cast iron contains generally above 2 per cent graphite and less than 1.5 per cent combined carbon, and may be considered as an impure steel mechanically mixed with graphite. Its other impurities are mainly Si, S, P, and Mn. Silicon and sulphur can be regulated in the blast and remelting furnaces. Phosphorus cannot be regulated, and all that is in the ore goes into the cast iron or pig. Manganese can be controlled to some extent.

Silicon occurs in foundry pig up to 3.75 per cent. It facilitates the separation of carbon as graphite, and thus increases softness, soundness, fluidity, and decreases shrinkage, but above 3 per cent it may cause hardness by uniting with the iron to form a silicide. In cast iron to be used for making steel, silicon should not be over 1 per cent, in order to limit the quantity of slag and the loss of iron in the steel furnace. It also tends to cause the austenite to decompose into ferrite and graphite instead of into ferrite and cementite down to the eutectoid (not eutectic) point ( $H$  in the equilibrium diagram, Fig. 11). It should be remembered that the austenite in cast iron acts the same as in steel as the temperature falls.

Sulphur favors the retention of combined carbon more powerfully than silicon favors the opposite effect, hence it tends



to produce a hard, brittle iron. It also forms a sulphide of iron,  $\text{FeS}$ , which makes the iron very tender at a red heat ("red-short"), and thus tends to cause cracks in castings, due to shrinkage when prevented by the mold. Sulphur comes mainly from the fuel, though sometimes from the ore also. It can be counteracted by manganese, which has a greater affinity for sulphur, forming a sulphide that is much less injurious than  $\text{FeS}$ , which will not be formed if there is sufficient manganese. Hence sulphur is often limited, in specifications for castings, to not over 0.10 per cent and sometimes to not over 0.05 per cent, and it is required that the Mn must be not less than three times the S. For steel making, some of the S in the cast iron may be eliminated by one of the basic processes.

Phosphorus in the ore is not eliminated in the blast furnace, and as it is very objectionable in steel, it must not exceed 0.05 per cent in ore having 50 per cent iron or 0.10 per cent in cast iron used for making steel by the acid process, but may be higher if the basic process is used, which eliminates most of the phosphorus. By thermo-electric smelting, with a slag rich in lime, the phosphorus may also be removed.

Phosphorus in itself tends to increase combined carbon in cast iron, but it also makes the melted iron more fluid, and so lengthens the time of solidification, thus indirectly favoring the formation of graphite. This indirect effect, when there is high silicon, makes phosphorus, on the whole, favor the formation of graphite; but high phosphorus and low silicon favor the production of white iron. It makes the iron brittle when cold.

Manganese increases combined carbon, but not to the extent that sulphur does; and so, to the extent that it combines with sulphur, it lowers the combined carbon. There should be enough manganese to prevent the formation of  $\text{FeS}$ ; any greater amount increases combined carbon. It thus counteracts the bad effects of sulphur, but it makes the casting hard and difficult to machine. Graphite increases the workability, not only because it is soft, but because it acts as a lubricant for the cutting tool.

It is thus seen that the impurities in cast iron are mainly of importance in their effect upon the form that the carbon assumes.

**7. Shrinkage.**—Cast iron, as its name implies, is always in the form of castings made by pouring the melted metal into a mold. In cooling, the metal solidifies and shrinks, the shrinkage depend-

ing largely upon the proportion of graphite. When the eutectic freezes, it breaks up into a solid solution of carbon in iron (or carbide of iron in iron) and graphite, that is, into saturated austenite and graphite; or into austenite and cementite, the latter later breaking up into graphite and ferrite. In the molten metal the carbon is in solution, and hence does not increase the volume; but when the carbon separates as graphite, as it does in gray iron, it is the same as introducing a new substance, and the volume is increased. There is therefore an initial expansion when the graphite separates, followed by a gradual contraction in cooling further. Any substance, therefore, which favors the formation of graphite (such as Si), and any substance which favors fluidity (such as Si and P), produces sharp castings with relatively little shrinkage, while any substance that favors the crystallization of the carbide as white iron has the opposite effect. With much graphite the shrinkage may be very small. The allowance for shrinkage of gray cast iron is about one-eighth of an inch per foot; white cast iron, and steel, shrink about double as much as gray iron.

As shrinkage is less the slower the cooling, and as large castings cool more slowly than small ones, shrinkage decreases as the size of the casting increases.

It is obvious that, in designing castings, certain principles must be observed, with the object of lessening the internal stresses produced in cooling, and thus increasing the strength of the casting. These rules are:

(1) Avoid sharp corners, especially re-entrant angles, by rounding the corners off with curves of considerable radius, even if these have to be machined down afterward. In cooling, the outside cools first, while the inside remains hot or molten, and therefore heat is conducted outward. The flow of heat to the re-entrant angle will obviously be more free than to the surface on either side of it, and the isothermal lines will be bent toward the angle instead of being parallel to the surface. The crystals grow at right angles to the surface, and slower along a line bisecting the re-entrant angle than perpendicular to the adjacent surfaces. There will thus be initial tension at the angle and inward from it. Projecting corners are not so bad.

(2) Avoid sudden changes in thickness; and do not connect parts of different thicknesses, as a thin rim of a wheel to thick spokes, except by a gradual curve; for the cooling will be more

rapid the thinner the metal, and hence the thicker metal, in cooling and shrinking after the thinner metal has become cold, will cause large internal stresses and perhaps cracks. If sudden variations in thickness cannot be avoided, it may be necessary to provide special means for cooling the thicker sections rapidly, so that internal stresses may be avoided or reduced. This may be done by chilling the thicker sections, by metallic pieces in the mold, or by a *water gate*, which is a column of porous material in the mold, down which water may be poured.

(3) Avoid shapes in which the ends of the casting cannot contract without crushing the mold, which tends to produce stresses in the intermediate parts. Arrange if possible so that all parts of the casting may shrink without restraint and as uniformly as possible.

The subject of foundry practice and the principles involved in making castings of cast iron or steel should be understood by the structural engineer, and he is strongly recommended to read Chap. X of Stoughton's "Metallurgy of Iron and Steel," where other references are given; and also the chapters on cast iron in Johnson's "Materials of Construction," and in Mills' "Materials of Construction."

**8. Defects in Cast Iron** (see Chap. XIII of Stoughton).—The defects in cast iron may be due to improper methods or carelessness in manufacture, or to excessive amounts of impurities; or there may be defects developed in the process of casting. The former may be guarded against by limiting the amount of impurities, and requiring care in manufacture. Defects of the latter class are: (1) *Cracks* (checks) due to errors in designing, which do not allow properly for contraction, and are especially likely to occur if the shrinkage is large or the sulphur high, since this element not only favors shrinkage, but also makes the iron red-short, or likely to crack when at a red heat. They can generally be detected by inspection or by the sound of the casting when struck with a hammer. Car wheels are periodically tested in this way. (2) *Segregation*, or concentration of the impurities at one place. This generally occurs at the point which freezes last, since each layer which solidifies rejects some of its impurities, which pass into the still liquid metal. Segregation is less if the sulphur and phosphorus are low, especially if the carbon is also low. (3) *Blow holes* may occur, especially if the mold is not properly vented so that the gases may escape. (4) *Porous* or

*spongy structure* may occur if the solidification does not proceed uniformly.

**9. Grades of Pig Iron.**—Pig iron is usually classified according to the purpose for which it is to be used, as follows:

Bessemer pig; for making steel by the acid Bessemer<sup>1</sup> or acid open-hearth process;

Basic pig; for making steel by the basic open-hearth process;

Malleable pig; for making malleable cast iron (see Chap. VI);

Foundry pig; for making gray iron castings;

Forge pig; for making wrought iron.

The chemical composition of these grades is usually kept within the following limits, according to Mills:

	Silicon, per cent	Sulphur, per cent	Phosphorus, per cent
Bessemer pig.....	1.0 to 2.0	not over 0.05	not over 0.10
Basic pig.....	under 1.00	under 0.05	not specified
Malleable pig.....	0.75 to 2.00	not over 0.05	not over 0.20
Foundry pig.....	1.50 to 3.00	not over 0.05	0.50 to 1.00
Forge pig.....	under 1.50	under 0.10	under 1.00

**10. Strength of Cast Iron.**—The *tensile strength* is small, and depends largely upon the proportions of graphite and cementite, being greatest, according to Howe, when the carbon in cementite is about 1.2 per cent and the graphite 2.8 per cent in a total of 4 per cent carbon. The specifications of the A.S.T.M. for gray iron castings provide the following *minimum ultimate tensile strength* in pounds per square inch:

For light castings, having any section  $< \frac{1}{2}$  inch thick; 18,000

For heavy castings, having no section  $< 2$  inches thick; 24,000

For medium castings, not included in the above; 21,000

The stress-strain diagram in tension is convex upward, the value of  $E$  continuously decreasing from the origin. The initial value is about 30,000,000 for hard iron, 24,000,000 for average and 14,000,000 for soft iron; but at 10,000 pounds per square inch these values are reduced by one-half or more. The reduction of area is scarcely appreciable, and the elongation rarely over 3 or 4 per cent. Figure 28 gives some typical diagrams, but the material is very variable.

<sup>1</sup> The basic Bessemer process is not used in the United States.

The *compressive strength* of cast iron is large, varying greatly with the quality. Short specimens have ultimate strength from about 35,000 pounds per square inch for soft gray iron to over 100,000 for hard white iron. The diagram shows that  $E$  is more nearly constant, up to a certain point, than in tension, especially for hard iron, the initial value being about the same as for tension.

In *flexure*, the actual extreme stress, even for small loads, is not the same as the modulus of rupture computed from the load by

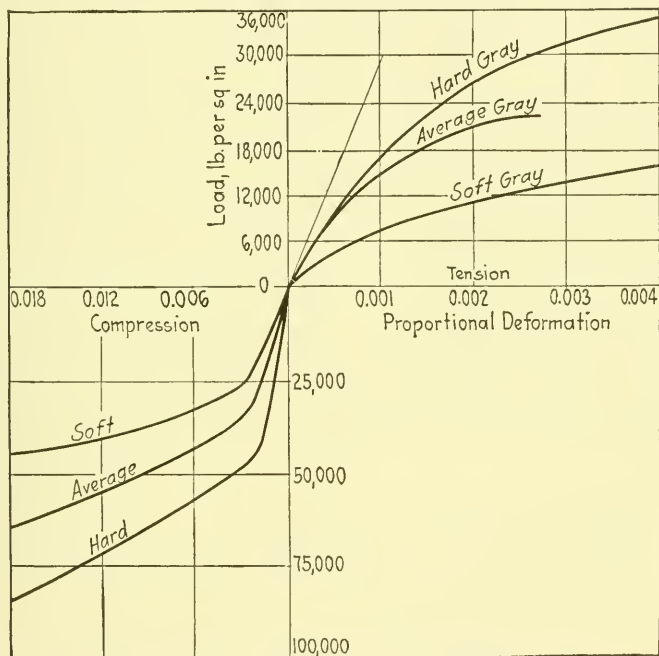


FIG. 28.—Stress-strain diagrams for cast iron. (Mills.)

the usual formula, because  $E$  is not constant, or precisely the same in tension as in compression for all loads. The computed modulus of rupture generally varies from 1.5 to 2.25 times the tensile strength, in solid rectangular sections, averaging about 1.8; Lanza<sup>1</sup> gives an average of about 2: for other forms of section it is less. Tests of round bars with spans of 12, 18, and 24 inches, of gray cast iron, gave the computed modulus of rupture varying from 1.63 to 1.87 times the tensile strength.<sup>2</sup> These relations are

<sup>1</sup> See LANZA'S "Applied Mechanics," p. 378.

<sup>2</sup> A.S.T.M. vol. 10, p. 299.



important, because it is often necessary for the structural engineer to compute cast iron in flexure, as, for instance, in column bases. Computed by the flexure formula, the ultimate modulus of rupture will vary from about 39,000 for light gray iron castings to over 50,000 for heavy castings.

Specifications of the A.S.T.M. for gray iron castings prescribe a flexural test of a round bar, the so-called "arbitration bar," which is  $1\frac{1}{4}$  inches diameter at one end and  $1\frac{3}{16}$  inches at the other (to allow easy removal from the mold), and 15 inches long. Tested on a span of 12 inches, such a bar must sustain a minimum central load of 2,500, 2,900, and 3,300 pounds for light, medium, and heavy castings respectively, and must deflect at least 0.1 inch at the center; cast iron for locomotive cylinders must sustain at least 3,200 pounds with a deflection of 0.09 inch. These loads correspond to values of the modulus of rupture of about 39,100, 45,400 and 51,600, respectively.

**11. Allowable Stresses.**—Owing to its unreliability, variability, and brittleness, the factor of safety for cast iron should be larger than for wrought iron or steel. As cast iron is now generally prohibited for bridges, there are no specifications for allowable stresses for that material in such structures. For buildings, the specifications of the National Board of Fire Underwriters allow:

	POUNDS PER SQUARE INCH
Cast iron in compression, short pieces.....	16,000
Cast iron in shear.....	1,500
Cast iron in flexure, compression side.....	16,000
Cast iron in flexure, tension side.....	2,500
Cast iron in columns $9,000 - 40\frac{l}{r}$ (Maximum $\frac{l}{r} = 60$ )	

The cast iron to which these specifications refer is described as follows:

Cast iron shall be of good foundry mixture, producing a clean, tough, gray iron. Castings shall be free from serious blow-holes, cinder spots, and cold shuts. Transverse tests on cast iron shall be made upon the  $1\frac{1}{4}$ -inch diameter "Arbitration Bar" of the American Society for Testing Materials. The bar to be supported on 12-inch centers, loaded at the middle, and in no case shall it test at less than 2,900 pounds. Tensile tests optional.

Water pipes of cast iron are exposed to hoop tension. Thickness of such pipes is standardized for various diameters and heads.

Thus, the A.S.T.M. specifies for a head of 400 feet, or a pressure of 173 pounds per square inch for a 48-inch pipe, a thickness of 1.96 inches, which gives a tensile stress of about 2,100 pounds per square inch. In such a pipe the hoop tension may exceed that due to the static head, due to *water hammer*. The stress allowed gives a factor of safety of at least 9.

**12. Protection of Cast Iron.**—Water pipes are protected by dipping them, after heating to 300° F., in a varnish made of hot coal tar pitch and oil, to give them a smooth surface and protect against corrosion. Structural cast iron should be painted or enclosed in fireproofing.

**13. Semi-steel** is a product made by melting a mixture of from 20 to 50 per cent of steel with pig iron. It is therefore a cast iron of low carbon content and high strength, and is sometimes used for castings requiring extra strength. It is neither steel nor cast iron.

## CHAPTER V

### WROUGHT IRON

1. Wrought iron, technically, is the product of the puddling furnace. It does not necessarily differ chemically from low-carbon steel except that it has mixed with it, in layers or threads, 1 to 3 per cent of slag. The difference between it and low-carbon steel is in the process of manufacture, wrought iron being formed by the aggregation of pasty masses, while all steels are formed in a molten mass.

In the puddling furnace, pig or cast iron and iron ore or scrap or mill scale are melted on the hearth of a furnace lined with iron ore or some form of iron oxide. The slag is basic, and the impurities of the cast iron are reduced and largely eliminated by the iron oxide of the slag and of the fettling or lining, and the oxygen of the furnace gases. The nearly pure iron solidifies in pasty masses, its melting point being higher than the temperature of the furnace. The pasty masses, dripping with slag, are removed by the puddler, and either squeezed or hammered, and then rolled into flat bars called "muck bars." These bars are cut into short lengths, piled in layers, generally all in the same direction but sometimes with the bars alternating in direction, and the pile is heated and rolled into "merchant bar." This operation is repeated once or twice.

It is customary to speak of wrought iron as fibrous. It is, however, not fibrous by structure, but only on account of the layers or threads of slag running in the direction of the rolling, which in a sense justifies the term. Figure 29 is a micrograph of wrought iron, in longitudinal section, showing ferrite and slag.

Wrought iron is practically pure ferrite, with a small amount of impurities, and with some slag mechanically mixed with it. There is some pearlite. The carbon is generally below 0.12 per cent. It is much lower in manganese than steel, this being perhaps the best method of distinguishing the two chemically, for it has not been recarburized as steel has. It has the advantage of being free from blow-holes. It generally contains

more phosphorus than steel, but this element appears to be less objectionable than in steel. It generally has lower ultimate tensile strength, elastic limit, elongation, and reduction of area than soft steel, and under compression a short prism is more liable to crack. It does not harden when cooled suddenly from a high heat. It can be welded easily.

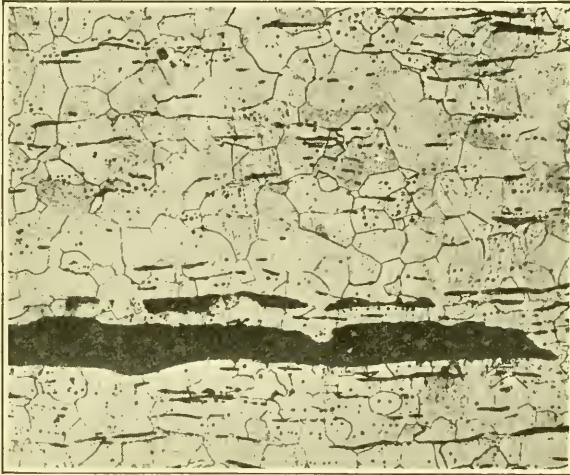


FIG. 29.—Wrought iron; longitudinal section, showing ferrite and slag. Magnified 100 diameters.

**2. Use of Wrought Iron.**—Wrought iron was formerly the principal material used for metal structures, cast iron having been used for certain parts such as compression pieces and occasionally for beams. Most metal bridges built more than 35 years ago are of wrought iron, and many of these are still in use. If the practice of the structural engineer is extensive, he is sure to meet them. Since the perfecting and cheapening of the methods of making reliable steel of any desired grade, even the softest, steel has practically entirely superseded wrought-iron for structures. Its principal uses now are in cases where its superior welding qualities are useful, as for certain rods and bars, and for welded pipe and boiler tubes; and in cases where its (supposed) greater resistance to corrosion is valuable, as for wire, stay bolts, tanks and other purposes.

**3. Strength of Wrought Iron.**—The *tensile strength* is greatest in the direction of rolling unless the muck bars have been *cross-piled*, when it may be about equal in both directions. Generally

the strength at right angles to the direction of rolling is about three-fourths that in the direction of rolling.

The specifications of the A.S.T.M. prescribe the following requirements for (A) staybolt, (B) engine bolt, and (C) extra refined wrought-iron bars, made from all-pig-puddled iron, without any admixture of iron scrap or steel, and for plates of class A' = from puddle bars made wholly from pig iron and such scrap as emanates from rolling the plates, and class B' = from puddle bars made wholly from pig iron or from a mixture of pig iron and cast-iron scrap, together with wrought-iron scrap:

	A	B	C	Wrought-iron plates			
				Class A'		Class B'	
				Width, 6-24 inches	Width, 24-90 inches	Width, 6-24 inches	Width, 24-90 inches
Tensile strength, $T$ , pounds per square inch.....	48-52,000	.....	48-54,000	49,000	48,000	48,000	47,000
Section $\leq 1\frac{1}{4}$ square inches.....	.....	49-53,000	.....	.....	.....	.....	.....
Section $> 1\frac{1}{4}$ square inches.....	.....	47-53,000	.....	.....	.....	.....	.....
Yield point, by drop of beam, pounds per square inches.....	0.6T	.....	.....	26,000	26,000	26,000	26,000
Section $\leq 1\frac{1}{4}$ square inches.....	.....	0.6T	0.6T	.....	.....	.....	.....
Section $1\frac{1}{4} - 4$ square inches.....	.....	0.55T	0.55T	.....	.....	.....	.....
Section $> 4$ square inches	.....	0.5T	0.5T	.....	.....	.....	.....
Minimum elongation in 8 inches, per cent....	30	25	25	16	12	14	10
Minimum reduction area in 8 inches per cent..	48	40	37	.....	.....	.....	.....

The value of  $E$  is generally from twenty-six to twenty-eight million pounds per square inch.

The tensile strength  $T$  is greater the finer the grain, and the grain will be finer the greater the amount of mechanical work put upon the material and the lower the temperature at which that work is finished. For the greatest fineness and strength, the rolling should not cease till the temperature has fallen about to the line  $LH$  in Fig. 11; i.e., to about  $700^{\circ}$  C. There is therefore an increase of strength the greater the proportionate reduction of area from the pile to the finished bar; not, as is often stated, the



smaller the bar. Commander Beardslee found the following results:

When the bar was 6.62 per cent of the pile in area,  $T$  was 56,543  
pounds per square inch

When the bar was 11.63 per cent of the pile in area,  $T$  was 51,848  
pounds per square inch

with intermediate values for intermediate percentages. The effect of reduction of area in rolling is more pronounced on the elastic limit than it is on the ultimate strength. Rods  $\frac{3}{8}$  inch in diameter may show a true elastic limit as high as 35,000, while for rods 2 inches or more in diameter it may be as low as 25,000. The yield point of wrought iron is apt to be farther above the

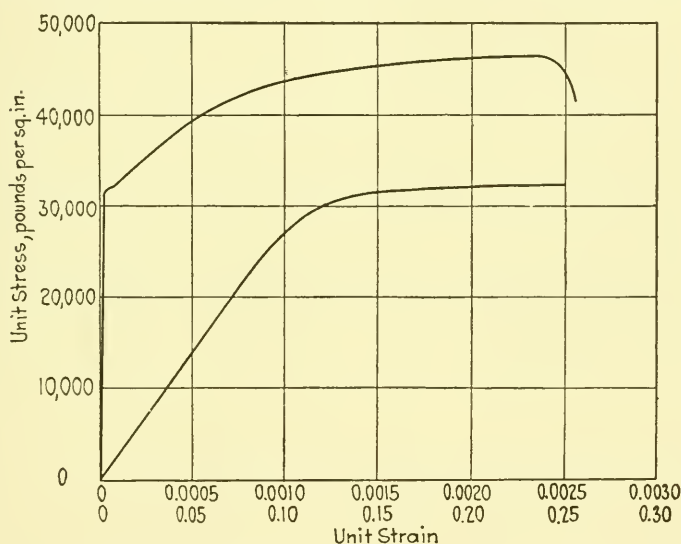


FIG. 30.—Stress-strain diagram for wrought iron (two different scales).

true elastic limit than in steel, sometimes the difference being nearly 10,000 pounds per square inch.

Figure 30 is a typical stress-strain diagram.

Cold working and overstressing above the elastic limit raises the latter, and also the ultimate, especially if the piece is allowed to rest.

Heating to a high temperature favors coarse crystallization, especially if the piece is maintained a considerable time at that temperature. Slow cooling also favors coarse crystallization. Mechanical work during cooling breaks up the coarse crystals and

produces fine grain. The melting temperature of wrought iron is very high, and for welding it must be heated moderately close to that temperature; hence when welding is done the part heated, not only at the weld, but all that is heated above the critical range, should be refined under the hammer to below the critical range. Often this is done at the weld, but a weak spot is left a short distance from it, where the metal has not been worked.

The *compressive* strength of wrought iron in short lengths is from about 45,000 to 60,000 pounds per square inch. The strength of long columns, as in the case of steel, will be given by a formula in which the numerator should be but little above the yield point, or from 25,000 to 35,000 pounds per square inch. The ultimate is for this case a little above the yield point.

The *shearing* strength of wrought iron is frequently assumed as 80 per cent of the tensile. It is less on a plane parallel to the rolling, or in the direction of the *fibers*, than on a plane perpendicular thereto, unless the muck bars were cross-piled. The difference between the strength in the two directions is sometimes 100 per cent. Mills states the shearing strength parallel to rolling 20,000 to 35,000 pounds, and on a transverse plane 30,000 to 45,000 pounds. Longitudinal shearing should be avoided, or the muck bars should be cross-piled; in the latter case the strength may be equal in the two directions but less than in a transverse direction without cross-piling.

## CHAPTER VI

### MALLEABLE CAST IRON

1. This is white cast iron which has been made stronger and somewhat malleable, without fusion, by packing it in some pulverized material and annealing it at a temperature of about 700 to 900° C. for from 60 to 100 hours, or longer. It has the advantages of cast iron, in low fusibility and ease of casting into forms which could be forged of iron or steel with difficulty or not at all, and also the advantages of much greater strength, toughness, and ductility than is possessed by cast iron.

The weakness of cast iron is due to the fact that white cast iron is hard and brittle, while in gray cast iron the graphite is not uniformly disseminated through the mass, but occurs in comparatively large flakes which form points or planes of weakness. If there were less graphite and if it were uniformly disseminated, the metal would be stronger. In white cast iron the carbon is almost all in combination with the iron, and is uniformly disseminated, for each particle or molecule contains its proportion of carbon.

In making malleable castings, therefore, they are first made of white iron, and then annealed, by which the carbon is changed from the combined form to an amorphous graphitic form, not in flakes, but uniformly and finely disseminated through the mass, in which condition it is called "temper-carbon." A cast iron low in carbon is desirable, it is poured at a high temperature so as to be fluid, and it is cooled rapidly, to produce white iron. The silicon is comparatively low, to facilitate the formation of combined carbon, yet it must be high enough to allow the separation of carbon as graphite during annealing. Figure 31 shows a micrograph.

If the material in which the castings are packed during annealing is inert, such as sand or clay, the heat alone produces the change in the condition of the carbon. The cementite is broken up into ferrite and carbon. If the packing is cinder or slag, or iron ore, the metal is decarburized, some of the carbon

uniting with the oxygen of the packing; and generally there is some decarburization in any case unless the annealing is *in vacuo*. If the process is carried far enough, the iron may be converted almost into comparatively pure steel—ferrite with some temper carbon. Usually there is a thin skin of decarburized material, with a center of ferrite and temper-carbon, forming the *black-heart* malleable iron usual in this country. With longer heating, as in Europe, there is greater decarburization. The cooling after annealing is as slow as possible.

Even if malleable iron contains as much carbon as gray iron, the carbon as temper-carbon is much less weakening than as

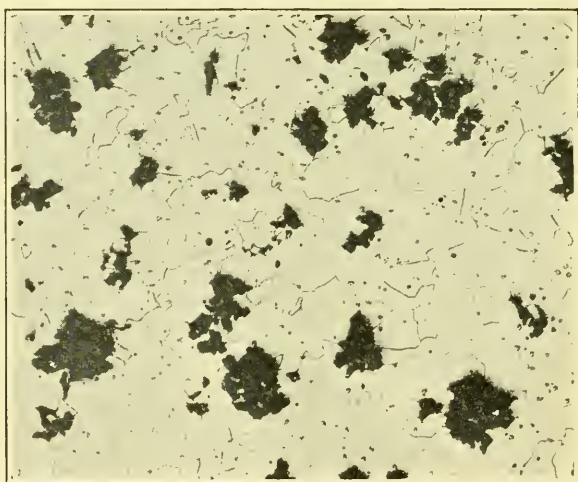


FIG. 31.—Malleable cast iron, showing ferrite white, and temper-carbon black

flakes of graphite; while the iron, not being in the form of a comparatively coarse matrix of ferrite and pearlite, is much tougher and less brittle.

**2. Strength and Other Properties.**—The A.S.T.M. specifications require for malleable castings a minimum *tensile* strength of not less than 45,000 pounds per square inch, and a minimum elongation in 2 inches of 7.5 per cent. The tensile strength is thus about double that of gray iron castings. The reduction of area may be as much as 12 per cent.

The crushing strength is no greater, and generally less, than that of cast iron, since the material more nearly approaches the character of wrought iron. The modulus of rupture is greater

than for cast iron, or from 65,000 to 90,000 pounds per square inch because the tensile strength has been so greatly increased. The toughness, or amount of energy absorbed up to tensile rupture, is very much greater than for cast iron, which may show 12 to 20 inch pounds per cubic inch, while malleable iron may show 250 to 500.

Malleable iron has thus considerable ductility and malleability. It may be flattened, or even bent double.

**3. Shrinkage.**—When the castings are first made which are later to be converted into malleable iron, since these castings are of white cast iron, the shrinkage is considerable, as almost no graphite forms. When, by annealing, the carbon in the cementite or white iron is converted into finely divided temper-carbon, there is an *expansion* which is about the same as what would have occurred if the graphite had been precipitated originally. This property must be allowed for in designing malleable-iron castings.

**4. Uses.**—Malleable iron has a large use for castings whose shape is such that they cannot well be forged, but which must be stronger and tougher than gray cast iron, and for pieces subject to tension. It is used for many parts of railway rolling stock couplers, journal boxes, brake fittings, etc.; for pipe fittings many of which must be threaded; for parts of agricultural implements; for many machinery parts; for tools; for post caps, beam hangers, column bases, and other structural purposes. Cast steel could be used for most of these purposes, but is more expensive.

**5. Case-hardening.**—Analogous, but opposite, to the process of making malleable iron, is the process of case-hardening, by which a low-carbon steel, or wrought iron, is packed in a carbonaceous substance, such as coke, charcoal or other form, from which it absorbs carbon into the outer layers, which become case-hardened and form a coating of higher carbon steel. Such a steel, when quenched, will be hardened on the outside while the soft inside core will be little affected. The heating should be to above the point  $A_{c1}$ .



## CHAPTER VII

### STEEL

1. Steel results from the purification of cast iron, by which the carbon is reduced to the desired proportion, and other impurities either removed or reduced to proper limits; or by the addition of carbon to wrought iron. Beginning with pure iron, which can only be obtained in the chemical laboratory, as the carbon is increased a graded series of products results, from low-carbon steel to the cast irons, the latter varying in physical properties not so much in proportion to the amount of carbon, as according to the form in which it occurs. The steels also, as has been seen, vary greatly according to the constitution as well as the composition.

Formerly steel was intermediate between wrought iron and cast iron, and was made from wrought iron by adding carbon, by the cementation or by the crucible process. While these processes are still used for some kinds of steel, almost all steel is now made from cast iron by the Bessemer or by the open-hearth process, by which the carbon is first almost all burned out, and then enough is added to produce the grade of steel desired.

Some grades of steel, therefore, have no more carbon than wrought iron, and, like it, will not harden when suddenly cooled from a high heat. Other kinds, containing some alloying element, are, like cast iron, not malleable, except at certain temperatures.

Ordinary or *carbon steel* has but small amounts of other elements except carbon. *Alloy steels* are those to which more considerable quantities of other elements are added for the purpose of producing desired properties: they will be briefly referred to presently.

The logical line of division between steel and cast iron would seem to be at the maximum percentage of carbon which can exist in solid solution with iron above the critical temperature. This limit is at 1.7 per cent carbon. Below this, the carbon (or carbide) is in solid solution above the critical temperature and the material is homogeneous, while below the critical temperature the materials separate, while solid, into crystals of different characters, as explained in previous pages. Above the limit of 1.7 per cent *C*, the carbon (or carbide) crystallizes out in solidifying,

and the metal is not homogeneous, but consists of crystals of carbon or carbide in a mass of different constitution. Ordinarily steel has less than 1.7 per cent *C* and cast iron has over 2.5 per cent *C*; hence carbon steel usually contains 98+ per cent of iron, 1.7— per cent of carbon, and some impurities.

**2. Processes of Manufacture.**—The distinction between steel and wrought iron is not in composition, but in method of manufacture. Wrought iron is made by melting cast iron in the puddling furnace, exposing it to a flame which burns out the carbon, gathering the wrought iron into pasty masses and removing it, to have the slag squeezed out and to be rolled into bars. Steel is made, in the Bessemer and open-hearth processes, by burning out the carbon and casting the liquid steel in an ingot.

In the *cementation* process, wrought-iron bars are packed in boxes, surrounded by carbonaceous material. Under the high heat of the furnace, the carbon penetrates into the bars at the rate of about  $\frac{1}{8}$  inch in 24 hours, the slag in the wrought iron causes the evolution of gas, and the bars, when removed, are covered with blisters, so that this steel is known as "blister steel." It is then melted.

In the *crucible* process, wrought iron and carbon (charcoal) are placed in crucibles and melted into steel. Other elements may be added. In England it is customary to remelt blister steel.

These last two processes are in general only used for making tool steel.

The Bessemer and the open-hearth process may be *acid* or *basic*. These processes have for their object to burn out the carbon and to eliminate the other impurities so far as possible, or to reduce them to proper limits.

In the acid Bessemer process, air is blown through the molten cast iron, which is run into the converter from the mixers. The oxygen of the blast oxidizes the silicon, manganese, carbon, and some of the iron. An acid slag forms on top of the iron, and the lining of the converter must be acid, in order that the acid slag may not attack it and quickly eat it away. Sulphur and phosphorus may be oxidized, but, if they are, the acid slag will not take them up since they are acid themselves, and so they will be deoxidized and remain in the steel. It is therefore essential, in this process, to use a cast iron with no more of these elements than is allowable. When the carbon is nearly all burned out, a recarburizer, ferromanganese, is added, as it is cheaper to burn out almost all the carbon, silica and manganese, and then add the

necessary amount than to try to stop the burning at the proper point, which would be difficult.

In the basic Bessemer process lime is added to the molten metal, in order to form a basic slag, and the lining must be made basic in order that it may not be attacked. The basic slag will take up some of the sulphur and much of the phosphorus, which may therefore be removed by this process.

The difference between the acid and basic processes is therefore that in the former the slag is acid (that is, with excess of  $\text{SiO}_2$ ), the lining is acid (silicious), and sulphur and phosphorus are not eliminated; while in the basic process lime is added in sufficient quantity to more than satisfy the silica, the lining is basic (dolomite or magnesite and tar), the sulphur and phosphorus are taken up by the basic slag, so that a cast iron may be used with more of these elements than in the acid process. The lining plays no part in the process of conversion.

In the Bessemer process the charge is comparatively small (10 to 15 tons), and the time of a *blow* short (about 10 minutes). The air blown through the metal leaves some dissolved gases, and some of the iron is oxidized, so that it is necessary to add a proper amount of recarburizer containing not only the desired amount of carbon, but also manganese and silicon, to deoxidize the iron, unite with what sulphur is left to form  $\text{MnS}$  instead of  $\text{FeS}$ , and remove the dissolved gases.

In the *open-hearth process*, the oxidizing flame, produced by burning gas with air, sweeps over the surface of the molten metal on the hearth of the furnace instead of being blown through it. The charge is relatively large (50 to 200 tons), the time of a heat long (about 10 hours). The process may be acid or basic. Steel scrap, or iron ore, or both, are often added to the charge of cast iron. The process can be stopped at the proper point more easily than in the Bessemer process, yet it is customary to carry it further and to add a recarburizer, but a smaller amount than in the Bessemer process, since there are less dissolved gases and often the carbon has not been so completely burned out.

In the basic process, whether Bessemer or open-hearth, the recarburizer cannot be added directly to the bath, for if that were done the basic slag would take silicon from the recarburizer and throw phosphorus back into the metal. The recarburizer must therefore be added after the slag has been drawn off, generally to the metal as it flows from the furnace, or in the ladle into which the finished metal is run. Even then, in the basic process some

slag may be carried over, and some phosphorus thrown back into the metal.

In the Bessemer process no fuel is used, the impurities of the bath furnishing the fuel; in the open-hearth process the fuel is a pre-heated mixture of air and gas. In the Bessemer process none of the charge is melted in the converter, but is run into it melted from the mixers; in the open-hearth process some or all of the charge of pig iron, scrap, and ore is melted in the furnace.

The basic Bessemer process is not used in the United States.

**3. Impurities.**—The most injurious elements in steel are sulphur and phosphorus.

*Sulphur* in the ore can be eliminated or reduced by roasting the ore. What remains, and what comes from the fuel, can be controlled in the blast furnace, so that there should be little in the pig. This can be partly eliminated in the basic steel process, and partly counteracted by adding manganese in the recarburizer. Sulphur in iron or steel exists as sulphide of iron or of manganese, and as manganese has greater attraction for it than iron has, manganese will take it from iron.

The injury done by sulphur arises from the fact that manganese sulphide exists as small drops or masses which are liquid or pasty at the rolling temperature of steel, so that at this temperature the sulphur makes the steel tender and liable to crack, or "red-short." Iron sulphide is more injurious than manganese sulphide because, instead of coalescing, it spreads out in sheets, and is more weakening. Manganese is therefore added to steel to take the sulphur from the iron, and the manganese sulphide normally goes off in the slag, though some may remain. Manganese also gets rid of the oxygen, by being oxidized and passing off with the slag in the acid process. In cast iron or cast steel the effect of sulphur is to make the castings liable to crack when solidifying, especially if the mold is such that shrinkage cannot occur without compressing the sand, so that it causes shrinkage tensile stresses.

It is desirable, therefore, to have all the sulphur in the form of manganese sulphide, and there should be manganese enough to ensure this result, with some to spare. Since the percentage of Mn in MnS is  $55\frac{1}{32}$  times the percentage of sulphur, it is considered that in steel the percentage of manganese should be three to four times the percentage of sulphur.

*Phosphorus* would do little harm if small in amount and uniformly disseminated, but it tends to become localized or segregated as phosphide of iron.



Phosphorus is injurious because it makes steel brittle when cold, or "cold-short," especially if subjected to shock. This brittleness increases with the carbon. Phosphorus cannot be controlled in the blast furnace, but can be partly removed by the basic steel processes. As the basic Bessemer process is not used in the United States, it is essential that ore used here for making Bessemer steel should be low in phosphorus, the limit allowed being 0.05 per cent. Sulphur must also be low, as it is not removed in the acid Bessemer process, the usual limit in the cast iron used in this process being 0.2 to 0.7 per cent. In the basic Bessemer process, largely used in Europe, high phosphorus is desirable (from 1 to 3 per cent) as it forms an important part of the fuel. Ores of sufficient purity have hitherto been available in the United States, but are becoming scarcer. The basic process is less expensive than the acid process, because, although the basic lining costs more than the acid lining and does not last so long, the high phosphorus ores and scrap cost less than the ores required for the acid process.

Silicon is desirable in small amount, to get rid of blow holes.

**4. Comparison of Materials.**—When steel was first used to any considerable extent for structures, in the 1880's, many engineers were doubtful of it and thought it an unreliable material, preferring wrought iron, because the latter was softer and more ductile, because it was fibrous and therefore less likely to have a microscopic crack extend clear across the piece, and because it did not harden when suddenly cooled and was therefore less liable to injury in heat treatment or mechanical working. Since that time steels of all grades have come into use, soft and ductile if desired, and having strength considerably above that of wrought iron, thus effecting a saving of material. Wrought iron is now mainly used where welding is required, as it welds better than steel; and where special resistance to corrosion is desired, as many engineers believe that it resists corrosion better than steel. It is thus used for wire, for pipe, as a covering for roofs and sides of buildings, etc. Almost all structural work and machinery is made of steel.

When steel was first used for structures, it was only in structures of large size and in certain members, such as chords and eye bars, to attain economy by using a material of high ultimate strength and elastic limit, and so reducing weight. Steel was then much more expensive than wrought iron. Soon it was recognized that this high steel was not well suited to resist impact, and not a proper material for railroad bridges, and the



required strength was lowered while the ductility was increased. The extensive use of steel in structures did not commence till about 1890, when some railroads began to use it for spans of medium or short length. Schneider<sup>1</sup> gives the following table showing large bridges in which steel was used prior to 1899.

IMPORTANT BRIDGES OF STEEL, PRIOR TO 1899

Year of completion	River	Description	Ultimate strength, lbs. per sq. in.	Minimum elongation per cent
1874	Mississippi	Arches, St. Louis.....	100,000 min.	18
1880	Missouri	Plattsmouth Bridge.....	80,000 min.	12
1882	Missouri	Bismarck:		
		Comp.....	80,000-90,000	12
		Tension.....	70,000-80,000	18
1882	East River	Brooklyn Bridge, excl. wire.....	70,000 min.	
1883	Niagara	Cantilever.....	80,000 min.	15
1884	Susquehanna	B. & O. R. R.:		
		Comp.....	80,000 min.	15
		Tension.....	70,000 min.	18
1885	Arkansas	Van Buren Bridge:		
		Comp.....	80,000 min.	15
		Tension.....	70,000 min.	18
1885	Ohio	Ky. and Ind. Cantilever:		
		Comp.....	80,000 min.	15
		Tension.....	70,000 min.	18
1886	Harlem	Washington Arch, New York.....	62,000-70,000	18
1887	Missouri	Sibley Bridge, A. T. & S. F.:		
		Comp.....	75,000-85,000	18
		Tension.....	60,000-70,000	23
1888	Missouri	Omaha:		
		Comp.....	80,000 min.	15
		Tension.....	70,000 min.	18
1888	Mississippi	Cairo Bridge.....	67,000-75,000	20
1888	Ohio	C. & O. Ry. Cinn. & Covington:		
		Comp.....	64,000-72,000	19-17
		Tension.....	58,500-66,500	20-18
1890	Firth of Forth	Cantilever, Scotland:		
		Comp.....	76,000-83,000	17
		Tension.....	67,000-74,000	20
1890	Colorado	Red Rock Cantilever:		
		Comp.....	64,000-72,000	19-17
		Tension.....	58,500-66,500	20-18
1890	Mississippi	Merchants Bridge, St. Louis.....	67,000-75,000	20
1891	Ohio	Cantilever Highway, Cincinnati.....	62,000-72,000	22
1892	Mississippi	Memphis Cantilever:		
		Main trusses.....	69,000-78,500	18
		The rest.....	64,000-72,500	22
1893	Mississippi	Bellefontaine.....	62,000-70,000	22
1895	Delaware	P. R. R. Phila.:		
		Main trusses.....	62,000-70,000	22
		The rest.....	50,000-60,000	26
1897	Niagara	Double Deck Bridge.....	60,000-68,000	20
1898	Niagara	Highway Arch.....	60,000-68,000	20

<sup>1</sup> *Proc. A.S.T.M.*, 1902, p. 63.

By 1894, wrought iron shapes were practically unobtainable, and two grades of steel were in use; medium steel by those who advocated steel, and soft steel by those who perhaps preferred wrought iron, but being unable to obtain it, desired a material as nearly like it as possible. Present practice is quite uniform and has been standardized. The usual requirements will be given later.

Open-hearth steel is very generally considered superior to Bessemer steel mainly because the open-hearth process is more easy to control, because less recarburizing is required, and because open-hearth steel contains less gases, which are injurious. Bessemer steel is generally believed to be liable to be brittle and unsound, with less resistance to shock. Thus specifications for bridges, locomotives, cars, ships, axles, etc., require open-hearth steel; in steel buildings and rails, however, Bessemer steel is permitted (see table following).

MATERIALS SPECIFIED BY A.S.T.M.

Material	Process specified
Carbon steel rails (1914).....	Bes. or O.H.
O.H. steel girder and high tee rails (1921).....	O.H.
Low-carbon steel splice bars (1914).....	Bes. or O.H.
Medium-carbon steel splice bars (1914).....	O.H.
High-carbon steel splice bars (1914).....	O.H.
Extra-high-carbon steel splice bars (1914).....	O.H.
Quenched high-carbon steel splice bars (1921).....	O.H.
Quenched carbon-steel track bolts (1921).....	O.H.
Quenched carbon-steel track bolts nuts (1921).....	Bes. or O.H.
Quenched alloy-steel track nuts (1921).....	O.H. or el. furnace
Quenched alloy-steel track bolts nuts (1921).....	Bes. or O.H.
Steel track spikes (1918).....	Bes. or O.H.
Steel screw spikes (1921).....	Bes. or O.H.
Structural steel for bridges (1921).....	O.H.
Structural nickel steel (1921).....	O.H.
Structural steel for buildings (1921).....	Bes. or O.H.
Structural steel for buildings (rivets, pls. Ls over $\frac{3}{4}$ inch to be punched).....	O.H.
Structural steel for locomotives (1921).....	O.H.
Structural steel for cars (1921).....	O.H.
Structural steel for ships (1921).....	O.H.
Rivet steel for ships (1921).....	O.H.
Carbon steel bars for railway springs (1916).....	O.H., crucible, or el. furnace
Carbon steel bars for railway springs (special silicon req.) (1918).....	O.H., crucible, or el. furnace
Carbon steel bars for vehicle and automobile springs (1916).....	O.H., crucible, or el. furnace
Silica-manganese steel bars for automobile and railway springs (1916).....	O.H., crucible, or el. furnace
Chrome vanadium-steel bars for automobile and railway springs (1916).....	O.H., crucible, or el. furnace
Billet steel concrete reinforcement bars (1914).....	Bes. or O.H.

## MATERIALS SPECIFIED BY A.S.T.M.—(Continued)

Material	Process specified
Carbon-steel and alloy-steel blooms, billets and slabs for forgings (1921).....	O.H. or el. furnace
Carbon-steel and alloy-steel forgings (1921).....	O.H. or el. furnace
Quenched and tempered carbon-steel axles, shafts, forgings for locomotives and cars (1921).....	O.H. or el. furnace
Quenched and tempered alloy-steel axles, shafts, forgings for locomotives and cars (1921).....	O.H. or el. furnace
Carbon-steel forgings for locomotives (1921).....	O.H. or el. furnace
Carbon-steel cars and tender axles (1918).....	O.H.
Cold-rolled steel axles (1921).....	O.H. or el. furnace
Wrought solid carbon-steel wheels (steam railway) (1916)...	O.H.
Wrought solid carbon-steel wheels (electric railway) (1916)..	O.H.
Steel tires (1916).....	O.H.
Steel castings (1921).....	O.H. el. furn., cruc. or side-blow converter
Lap-welded and seamless steel boiler tubes for locs. (1921)...	O.H.
Lap-welded and seamless steel boiler tubes for stationary service (1918).....	O.H.
Welded and seamless steel pipe (1921).....	Bes. or O.H.
Automobile carbon and alloy steels (1921).....	O.H., crucible, or el. furnace
Boiler and fire box steel for locs. (1921).....	O.H.
Boiler rivet steel (1921).....	O.H.
Cold-drawn Bes. steel automatic screw stock (1914).....	Bessemer
Cold-drawn open-hearth steel automatic screw stock (1915).	O.H.
Commercial bar steel (1921).....	Bes. or O.H.

Acid and basic open-hearth steel are allowed in most specifications. Formerly many engineers preferred acid open-hearth, because of danger of dissolved gases and irregularities of structure due to recarburizing basic steel after it leaves the furnace; and because the acid process does not remove phosphorus, and therefore requires a better cast iron to start with, which was considered safer than to use a cast iron with more phosphorus and depend on removing it. In modern specifications both processes are allowed, but the allowed percentages of sulphur and phosphorus are specified, the phosphorus being lower in basic steel. For the cables of the Delaware River bridge at Philadelphia (1923) acid open-hearth steel is specified with the following requirements:

Carbon, not more than 0.85 per cent

Phosphorus, not more than 0.04 per cent,

Sulphur, not more than 0.04 per cent,

Tensile strength, per square inch of gross section, not less than 215,000 pounds,

Yield point per square inch of gross section, not less than 144,000 pounds,

Elongation in 10 inches while under tension, not less than 4 per cent.

The following gives the specifications for steel for the Sydney Harbor bridge (1922) for plates and shafts up to and including 1 inch in thickness:

SPECIFICATIONS FOR SYDNEY HARBOR RIVER BRIDGE

	Carbon steel	Nickel steel	Chrome-nickel steel
Minimum tensile strength, in pounds per square inch.....	62,000-70,000	85,000-100,000	85,000-100,000
Minimum yield point, in pounds per square inch.....	35,000	50,000	50,000
Percentage of elongation in 8 inches.....	1,500,000	1,600,000	1,600,000
	Tensile strength	Tensile strength	Tensile strength
Minimum reduction of area, percentage.....	44	40	30

*Trans. A.S.C.E. Vol. lxxvi, 1923, p. 1298. "The Study of Steels for Engineering Structures," by George K. Burgess.*

Crucible steel is the best and most expensive kind, and is used for tools, and sometimes for springs. It is the best, because, like cementation steel, it is made in closed vessels, out of contact with the air.

Steel refined in the electric furnace is also used for some purposes, especially in producing alloy steels and high-grade castings.

**5. Strength of Steel.**—The strength of steel is very different according to the composition and the constitution, that is, according to the percentage of carbon and other elements and the heat treatment and mechanical working to which it has been subjected. Some alloy steels have very great strength. Increase of strength is generally accompanied by decrease of ductility. Carbon produces the greatest increase of strength with the least decrease of ductility. Alloy steels will be referred to in Chap. VIII.

With increasing carbon, the tensile strength of normalized steel increases until the eutectoid composition is reached, about 0.85 per cent *C*, because, being 100 per cent pearlite the grain is finest at that composition. With less carbon there is more free ductile ferrite, and with more carbon there is more free brittle cementite. This explains why ductility decreases as carbon increases. The books referred to in Chap. I give formulae

for the increase of tensile strength due to each increase of 0.01 per cent of carbon and other elements, but this supposes the same heat treatment in all cases, or normalized steel. Notwithstanding the great variation in strength, it is remarkable that the modulus of elasticity of all grades of steel remains nearly constant at about 30,000,000 pounds per square inch.

Sauveur gives Fig. 31a to illustrate the relation between tenacity, elongation and pearlite, in annealed (pearlite) steels.

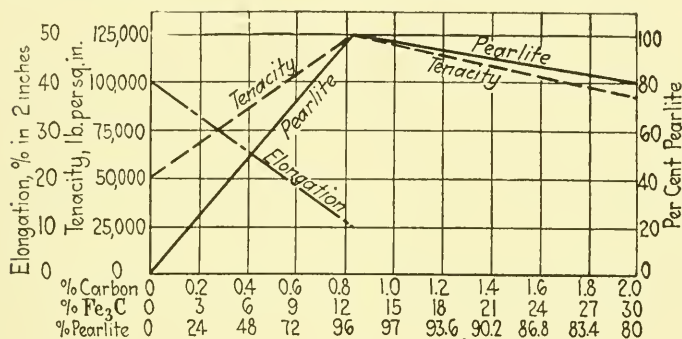


FIG. 31a.—Diagram showing the relation between the tenacity and ductility of annealed (pearlite) steels and the carbon content. (Sauveur, *Metallography of Iron and Steel*, 1912, p. v, 18.)

The *tensile* strength of carbon steel, depending on the different factors named, may be from 50,000 to over 150,000 pounds per square inch; and the stress-strain curve may vary from nearly that of wrought iron, with a well-defined yield point at which a large stretch occurs, to a line with practically no yielding at the so-called yield point (which in this case is very indefinite), curving gradually from the proportional limit up to the breaking strength.

Thus, Fig. 32 shows stress-strain diagrams for round rods before drawing into wire, with an area of 0.036 square inch, and the finished square wire with an area of 0.01 square inch. The rods themselves had probably been subjected to some mechanical cold working, of course, to reduce them to a diameter of 0.216 inch, so that the strength of the rods was above that of the same material normalized to remove the effect of mechanical working. The wire drawing was cold working, and the curves show the effect of this.

Steel does not stretch so uniformly over its entire length as wrought iron, under a tensile stress, but necks down more.



The table on pages 100 and 101 gives the physical properties of carbon steel used for various purposes, as specified in the 1921 Book of Standards of the A.S.T.M.:

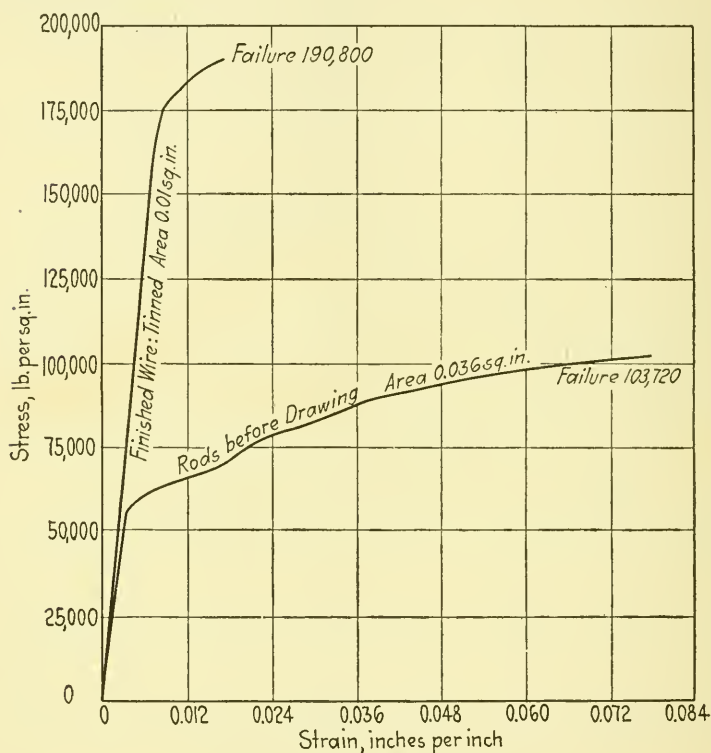


FIG. 32.—(Tests of Metals, 1890.)

The specifications for the Blackwell's Island bridge contained the following requirements (1904):

Material	Physical properties	Structural nickel steel	Structural carbon steel
Eye-bars unannealed	Ultimate tension	100,000	66,000
	Elastic limit	55,000	$\frac{1}{2}$ ultimate
	Elongation in 8 inches	<u>1,600,000</u>	<u>1,500,000</u>
		Ultimate tension	Ultimate tension
Pins unannealed	Ultimate tension	90,000	66,000
	Elastic limit	55,000	$\frac{1}{2}$ ultimate
	Elongation	20 per cent in 2 inches.	<u>1,500,000</u>
			Ultimate tension in 8 inches

The ultimate *compressive* strength of steel depends upon its composition and constitution, and the conditions of use. As shown in Chap. XXIII of the "Strength of Materials," it is impossible to conceive of the failure of an engineering material, or of any material, in pure compression unaccompanied by other kinds of stress, unless failure is taken to occur when the compressive deformation reaches a certain limit. Especially is this true of a material like steel. Short blocks in compression will fail by shearing or bulging; it is not pure compression, and deformation limits. A long column will fail by buckling of some part or of the column as a whole; in this case the ultimate strength depends upon the character of the material. If it is ductile, showing a large yielding at the yield point, in compression, the ultimate strength will be at the yield point or a little above. This, however, is a combination of compression and bending, accompanied by eccentricity of load. Even if the column as a whole remains straight until the yield point is reached, when that point is reached a flow of metal will take place, and some part, such as a thin outstanding leg, will be buckled, producing local eccentricity, and failure will occur, perhaps in the end by a tension break. As the material becomes harder, and shows less yielding at the yield point, the ultimate will rise, until for very hard material it may be considerably above the elastic limit; the yield point is here uncertain.

The *shearing* strength of steel is found by experiment to be not far from 0.8 the tensile strength.

The *torsional* strength is the shearing strength, but the formula for torsion will not give the ultimate strength equal to the true ultimate shearing strength because in torsion the shear is not distributed uniformly. This topic has been discussed in Art. 33 of Chap. X of the previous volume. The real stress at the outer fiber is less than that given by the torsion formula, by an amount depending upon the shape of the actual stress-strain curve under these conditions, and this is unknown.

The *flexural* strength is a combination of tension, compression, and shear, and involves the shape of the section. This subject has been referred to in Chap. X of the previous volume, and will be further treated in Vol. 3, in discussing the design of beams.

**6. Cold Bending.**—The ductility of steel is indicated by its ability to bend cold, without cracking, either flat upon itself or around a pin having a diameter depending upon the thickness of

SPECIFICATIONS OF THE A.S.T.M., 1921

Character and use of steel	Kind	Phos. maximum per cent	Sulphur, maxi- mum per cent	Tensile strength, pounds per square inch (u.t.s.)	Yield point, pounds per square inch	Elongation minimum, in 8 inches per cent	Elongation minimum, in 2 inches per cent	Reduction of area, minimum per cent
Structural steel for bridges	Structural.....	acid 0.06	0.05	55-65,000	0.5 (u.t.s.)	$\frac{1,500,000}{\text{u.t.s.}}$	22	
	Rivet.....	basic 0.04 acid 0.04 basic 0.04	0.045	40-56,000	0.5 (u.t.s.)	$\frac{1,500,000}{\text{u.t.s.}}$	22	
Structural steel for buildings	Structural.....	O.H. 0.06	.....	55-65,000	0.5 (u.t.s.)	$\frac{1,400,000}{\text{u.t.s.}}$	22	
	Bes. or O.H.	Bes. 0.10						
Structural steel for buildings	Structural.....	O.H. 0.06	0.045	40-56,000	0.5 (u.t.s.)	$\frac{1,400,000}{\text{u.t.s.}}$		
	Rivet.....							
Structural steel for locomotives.....	O.H.	O.H. 0.05	0.06	55-65,000	0.5 (u.t.s.)	$\frac{1,500,000}{\text{u.t.s.}}$		
Structural steel for cars	Structural.....	acid 0.06	0.06	50-65,000	0.5 (u.t.s.)	$\frac{1,500,000}{\text{u.t.s.}}$		
	Flange plates (for cold pressing).....	basic 0.04						
Structural steel for cars	Structural.....	acid 0.06	0.06	48-58,000	0.5 (u.t.s.)	$\frac{1,500,000}{\text{u.t.s.}}$		
	Rivet.....	basic 0.04						
Structural steel for ships	Structural.....	acid 0.04	0.04	45-60,000	0.5 (u.t.s.)	$\frac{1,500,000}{\text{u.t.s.}}$		
	Bes. 0.04							
Structural steel for ships	Structural.....	acid 0.06	0.06	58-68,000	0.5 (u.t.s.)	$\frac{1,500,000}{\text{u.t.s.}}$		
	Rivet.....	basic 0.04						
Carbon-steel rails.....	Structural.....	acid 0.06	0.045	55-65,000	0.5 (u.t.s.)	$\frac{1,500,000}{\text{u.t.s.}}$		
	Rivet.....	basic 0.04						
Carbon-steel rails.....	O.H.	O.H. 0.04						
	Bes.	Bes. 0.10						

Carbon-steel bars for railway springs	Grade A.....	O.H. crucible or el. furn.	0.05 0.05	0.05 0.05			
	Grade B.....						
Carbon-steel bars for Automobile springs.	Vehicle springs.....	O.H. crucible or el. furn.	acid basic 0.05 acid 0.05 basic 0.04	0.05 0.05 0.05 0.05			
	Automobile springs.	O.H.	O.H. 0.05	0.06			
Carbon-steel car and tender axles.....		O.H.	0.05	0.05	60,000 (elastic lim.)	18	35
Cold-rolled steel axles.....		O.H. el. furn.	0.05	0.05	.....		
Steel castings	Class A (not annealed unless specified)	O.H. crucible el. furn. or side-blow converter	acid 0.07 basic 0.06 acid 0.06 basic 0.05		0.45 (u.t.s.) 0.45 (u.t.s.) 0.45 (u.t.s.)	15 18 22	20 25 30
	Class B { Hard..... Medium..... Soft.....			0.06			
	Flange.....		acid 0.05 basic 0.04 acid 0.04 basic 0.035	0.05	0.5 (u.t.s.)	$\frac{1,500,000}{\text{u.t.s.}}$	
	Fire-box.....	O.H.		0.04	0.5 (u.t.s.)	$\frac{1,500,000}{\text{u.t.s.}}$	
Boiler and fire-box steel for locomotives							
Boiler rivet steel.....		O.H.	O.H. 0.04	0.045	0.5 (u.t.s.)	$\frac{1,500,000}{\text{u.t.s.}}$	
Billet steel concrete reinforce- ing bars	Structural.....				33,000	$\frac{1,400,000}{\text{u.t.s.}}$	
	Intermediate.....				40,000	$\frac{1,300,000}{\text{u.t.s.}}$	
	Hard.....	O.H.	O.H. 0.05		50,000	$\frac{1,200,000}{\text{u.t.s.}}$	
	Structural.....				32,000	$\frac{1,250,000}{\text{u.t.s.}}$	
	Intermediate.....	Bes.	Bes. 0.10		40,000	$\frac{1,125,000}{\text{u.t.s.}}$	
	Hard.....				50,000	$\frac{1,000,000}{\text{u.t.s.}}$	
Cold-twisted bars.....				.....	55,000	$\frac{5}{5}$	
Rail steel concrete reinforcement bars	Plain.....			.....	50,000	$\frac{1,200,000}{\text{u.t.s.}}$	
	Deformed.....			.....	50,000	$\frac{1,000,000}{\text{u.t.s.}}$	

the piece. The specifications of the A.S.T.M. for steel for various purposes are given in the following table:

STANDARD SPECIFICATIONS FOR COLD-BENDING TESTS OF VARIOUS STEELS

Character and use of steel				Cold bending		
				Angle, degree	Diameter <sup>1</sup>	
Bridge, building, car and locomotive structural steel				Not over ¾ inch thick.....	180	0
				¾ inch to 1¼ inches thick.....	180	t
				Over 1¼ inches thick.....	180	2 t
Rivet steel for bridges, buildings, boilers, ships, etc.....					180	0
Ship structural steel				Not over ¾ inch thick.....	180	t
				¾ inch to 1¼ inches thick.....	180	1½t
				Over 1¼ inches thick.....	180	2 t
Soft steel castings (specimens are 1 inch by ½ inch section).....					120	1 in.
Medium steel castings (specimens are 1 inch by ½ inch section)....					90	1 in.
Boiler and firebox steel.				Not over 1 inch thick.....	180	t
				Over 1 inch thick.....	180	2 t
Billet steel concrete reinforcing bars	Plain bars	Structural grade	Under ¾ inch thick.....	180	t	
			¾ inch thick or over.....	180	t	
		Intermediate grade	Under ¾ inch thick.....	180	2 t	
			¾ inch thick or over.....	90	2 t	
		Hard grade	Under ¾ inch thick.....	180	3 t	
			¾ inch thick or over.....	90	3 t	
	Deformed bars	Structural grade	Under ¾ inch thick.....	180	t	
			¾ inch thick or over.....	180	2 t	
		Intermediate grade	Under ¾ inch thick.....	180	3 t	
			¾ inch thick or over.....	90	3 t	
		Hard grade	Under ¾ inch thick.....	180	4 t	
			¾ inch thick or over.....	90	4 t	
Cold-twisted bars				Under ¾ inch thick.....	180	2 t
				¾ inch thick or over.....	180	3 t
Concrete reinforcing bars from rerolled rails	Plain bars	Under ¾ inch thick.....	180	3 t		
		¾ inch thick or over.....	90	3 t		
	Deformed bars	Under ¾ inch thick.....	180	4 t		
		¾ inch thick or over.....	90	4 t		

<sup>1</sup> Diameter of pin about which specimen of thickness  $t$  is bent.

If a piece of thickness  $t$  be bent around a pin of diameter  $d$ , and the inside of the piece around the bend does not change its length  $\frac{\pi d}{2}$ , the outside, which had originally this same length, "is stretched to a length  $\frac{\pi(d+2t)}{2}$ , or is stretched a distance  $\pi t$ ; or the stretch per unit length is  $\frac{2t}{d}$ . The severity of the test depends, therefore, upon the ratio  $\frac{t}{d}$ , and for uniformity this ratio should be constant. Generally, however, the material on the inside of the bend will be compressed.



**7. Steel at High Temperatures.**<sup>1</sup>—For some purposes, as for boilers, and for structural steel in a conflagration, the properties at high temperatures are of importance.

Figures 33, 34 from "Tests of Metals," 1888, shows the results for tensile strength. The strength slightly decreases up to about 300° F. for high carbon steels; at about 600° F. the strength is a maximum and some 10 to 15 per cent above that at normal temperature; from this point it decreases rapidly until at about 800° C. it is small, as might be expected.

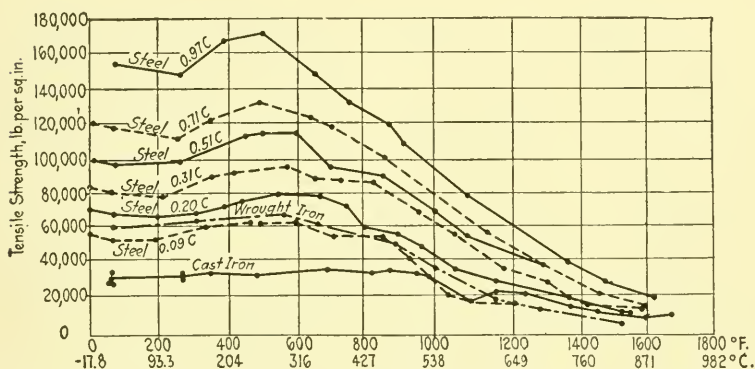


FIG. 33.—Variations in tensile strength of ferrous metals with temperature. (*Tests of Metals*, 1888.)

Cast iron is reduced in strength less than wrought iron or steel being not materially affected up to about 550° C., above which it falls, though not as rapidly as wrought iron or steel.

The modulus of elasticity decreases at high temperatures, as would be expected.

At a blue heat (500 to 600° F.) the ductility and toughness of wrought iron and steel is much reduced.<sup>2</sup> Bars which at normal temperatures could be bent back and forth 12 to 26 times through an angle of 45° first on one side and then on the other of the straight position, withstood at a blue heat only 1½ to 3 bends. The resistance to impact is also least at this temperature (see also Art. 12, Chap. III). Annealing removes these effects.

<sup>1</sup> JOHNSON'S "Materials," by Withey and Aston, Chap. XXVII: *Technologic Papers of the U. S. Bureau of Standards No. 219*; Watertown Arsenal Tests of Metals, pp. 243–323, 1888.

<sup>2</sup> STROHMEYER in *Proc. Inst. C. E.*, vol. 84, p. 114.

At very low temperatures the strength and the elastic limit are increased, while the ductility is decreased in even greater proportion. The resistance to impact appears to decrease as the temperature falls below normal; the number of blows required to break a wrought-iron axle at 100° F. being found to be 50 per cent greater than the number required at 0° F. Rails are supposed to be more brittle in very cold weather than in warm weather, there being more rail fractures in winter than in summer; though this might be due to the fact that the frozen roadbed is more rigid than in summer.

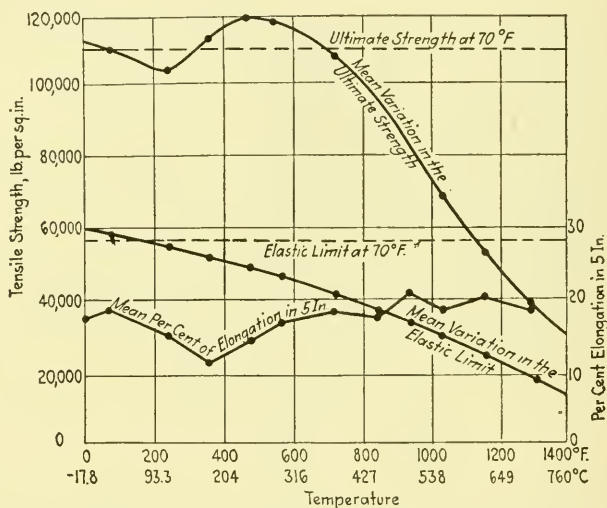


FIG. 34.—Grand mean curves from temperature tests on steel rods. Diameter of rods  $1\frac{1}{4}$  inch; of specimen 0.8 inch. Ten degrees of hardness, from 0.09 to 0.97 per cent C. (*Test of Metals*, 1888, p. 245.)

**8. Steel Castings.**—A steel casting does not receive the benefit of mechanical work, and is therefore likely to be more unsound, containing more dissolved gases, and more blow-holes, than forged or rolled material. The usual defects of castings are: (1) Blow-holes or gas bubbles; (2) a pipe, or cavity formed near the center or top of an ingot by the cooling of the outer surfaces and subsequent shrinkage as the remaining liquid cools; (3) ingotism, or the formation of large crystals, that is, coarse grain; (4) Segregation, or concentration of the impurities in the most easily fusible metal part which solidifies last; (5) Shrinkage stresses, which sometimes produce cracks.

*Blowholes* are liable to form in steel castings, especially in low-carbon steel, as air bubbles form in ice. They are particularly liable to occur in Bessemer steel, especially when the blast enters at the bottom and passes through the entire body of metal. Sometimes the blast enters at the side (side-blow converters) through the trunnions, and in this case blow-holes are less likely to occur. Manganese and silicon, added in the recarburizer, combine with the oxygen and go into the slag.

Blow-holes in ingots which are afterward worked, as by rolling, are flattened out, and the surfaces, if not oxidized, are welded together and the gas forced out. In castings there is no such action, and the blow-hole remains.

A *pipe* generally occurs near or at the top of an ingot, and the top is cut off and discarded in order to remove it. A steel casting generally has a *riser* or *feeder* which contains liquid metal that runs down and feeds any cavity that may tend to form. This riser is cut off after the casting has cooled. Cast iron does not have a pipe because it expands on solidification, as shown in Chap. IV.

*Ingotism* or large crystallization, is sure to occur in castings to some extent. It is especially likely to occur in harmful degree if the steel is cast at too high a temperature or is allowed to cool too slowly. It may be regulated to some extent by skilful treatment. It is corrected by mechanical work on rolled or forged material, but it remains in castings, though annealing under proper conditions may largely remove it.

*Segregation* to a certain extent cannot be prevented, though it is believed that the addition of aluminum will reduce it. It depends somewhat on the shape of the casting. A high casting temperature heats the mold, and results in slow cooling of the casting, with consequent coarse structure and perhaps segregation; hence the casting temperature should be low. This will also decrease the shrinkage stresses.

*Shrinkage stresses* will always exist, but they may be reduced by proper design of the castings. Annealing will largely remove them.

It is obvious that *steel castings are less reliable than forged or rolled material*. The specifications of the A.S.T.M. require, if Bessemer steel is used, that it shall be from a side-blow converter. Both acid and basic steel are allowed, although acid steel is generally thought to be better for castings than basic steel. This is

because acid steel starts with a pig lower in sulphur and phosphorus than basic steel, hence there is less segregation; and, because of the easier recarburizing of acid steel and the more uniform dissemination of the recarburizer, there is apt to be less danger of blowholes. Steel castings in which strength is important should always be annealed.

9. The table on preceding page, from paper by Dr. Geo. K. Burgess, Director of the U. S. Bureau of Standards, in the *Trans. A.S.C.E.*, 1923, gives some structural steel specifications.

## SOME STRUCTURAL STEEL SPECIFICATIONS

Type	Number	Process of manufacture	Phosphorus, maximum		Sulphur, maximum	Carbon, maximum	Nickel, minimum	Tensile strength in pounds per square inch	Yield point, minimum in pounds per square inch	Percentage elongation in 8 inches, minimum	Diameter, on 180° cold bend $\frac{1}{4}$ inch
			Acid	Basic							
A.S.T.M., bridges....	A 7-21	Open-hearth	0.06	0.04	0.05	....	....	55,000-65,000	0.5 tensile strength	27.3-18 <sup>1</sup>	1T 2T 3T
A.S.T.M., nickel....	A 8-21	Open-hearth	0.05	0.04	0.05	0.045	3.25	85,000-100,000	50,000	17.6-14 <sup>1</sup> R.A. = 25	1T 2T
A.S.T.M., buildings..	A 9-21	Open-hearth	0.06	0.06	....	....	....	55,000-65,000	0.5 tensile strength	25.4-18 <sup>2</sup>	0T 1T 2T
A.S.T.M., locomotives	A 10-21	Bessemer Open-hearth	0.10 0.05	0.06 0.05	.... 0.06	....	....	55,000-65,000	0.5 tensile strength	25.4-18 <sup>2</sup>	0T 1T 2T
A.S.T.M., cars.....	A 11-21	Open-hearth	0.06	0.04	0.06	....	....	50,000-65,000	0.5 tensile strength	30.0-18 <sup>1</sup>	0T 1T 2T
A.S.T.M., ships.....	A 12-21	Open-hearth	0.06	0.04	0.06	....	....	58,000-68,000	0.5 tensile strength	26.0-18 <sup>1</sup>	1T 1½T 2T
Silicon.....	.....	Open-hearth	0.06	0.04	0.05	0.40; Si 0.20-0.45 Mn < 1.00	0.20-0.45 Ni > 1.20 Si < 0.15 Mn < 0.30	80,000-95,000	45,000	20-16.8 <sup>3</sup>	1T 1½T
Mayari nickel chromium.	.....	Open-hearth	0.06	0.04	0.05	0.4; ....	....	85,000-100,000	50,000	18.8-16 <sup>3</sup> R.A. = 30	1½T
British bridges.....	15-1912	Open-hearth	....	0.06	0.06	....	....	62,720-73,920	.....	20-16 <sup>4</sup>	1½T
British buildings.....	15-1912	Open-hearth	....	0.08	0.06	....	....	62,720-73,920	.....	20-16 <sup>4</sup>	1½T
British ships.....	13-1910	Open-hearth	....	....	....	....	....	62,720-71,680	.....	20-16 <sup>4</sup>	1½T

<sup>1</sup> Based on  $\frac{1,500,000}{\text{Tensile strength}}$ .

<sup>2</sup> Based on  $\frac{1,400,000}{\text{Tensile strength}}$ .

<sup>4</sup> For steel below 0.375 inch in thickness.

<sup>3</sup> Based on  $\frac{1,600,000}{\text{Tensile strength}}$ .

*Trans. Am. Soc. Civil Engrs., 1923, p. 1295. "The Study of Steels for Engineering Structures," by George K. Burgess.*



## CHAPTER VIII

### ALLOY STEELS<sup>1</sup>

1. We have seen that steel is an alloy of iron with carbon and small quantities of other elements. The essential ingredient is carbon, and such steel is called "carbon steel"; the other ingredients being either those which cannot be eliminated, or which are purposely added in small quantities to produce some benefit without greatly changing the properties; as silicon to get rid of blowholes, and manganese to promote soundness by neutralizing the bad effect of sulphur and to eliminate oxygen. An *alloy steel* is one which owes its distinctive properties chiefly to the presence of an element or elements, other than carbon, or jointly to such element and carbon. If one other element is added, the alloy is a *ternary alloy*; if two are added, it is a *quaternary alloy*.

2. The principal ternary alloys are with nickel, manganese, chromium, tungsten, silicon, vanadium, molybdenum or titanium. The quaternary alloys are chrome-nickel, chrome-vanadium, and many others. There are alloys with more than four main ingredients, called complex alloys.

There are obviously innumerable alloys, depending on the proportions of the ingredients. The subject is very complicated, and rapidly changing as knowledge is gained. New alloys are being introduced, the character of some being kept secret.

3. The alloying elements in some cases form solid solutions with iron, while in others they form chemical combinations or liquid solutions which separate on freezing. A small proportion of an ingredient may produce very great effects, and in different proportions it may confer entirely different, or even opposite, qualities.

<sup>1</sup> The student should read the chapters on this subject in STOUGHTON, UPTON, MILLS and JOHNSON; also HIBBARD's short book on the "Manufacture and Uses of Alloy Steels." The paper by GEO. K. BURGESS, Director of the Bureau of Standards, in *Trans. A.S.C.E.*, pp. 1292-1315, 1923, should also be studied.

In most cases the alloying element depresses the critical temperature, at which austenite is transformed. Some have the effect of allowing austenite or martensite to be produced by slow cooling, while in carbon steel we have seen that these forms can only be obtained in the cold metal (if there is sufficient carbon) by very rapid cooling. Martensite produced by slow cooling is more stable than when produced by rapid cooling, for in the latter case the normal transformation is incomplete, having been arrested or prevented by the quenching. Nickel and manganese act in this way, and chromium to a less degree. This renders heat treatment easier in these alloy steels. If an alloy steel, cooled slowly, is martensitic in structure, heating and quenching it may make it austenitic, that is, may soften it, and the ductility and strength may be much increased.

If a hard steel is desired, it is an advantage if it can be produced by slow cooling, for heat treatment is thus rendered unnecessary. Alloy steels which become hard when cooled slowly are called "self-hardening steels."

4. The properties of alloy steels are very dependent upon heat treatment. By varying this treatment, a steel may be given a wide range of properties. The effect of heat treatment is illustrated by the following tests on pieces of steel which were all from the same heat but had been subjected to different heat treatment.<sup>1</sup>

Tensile strength	Elastic limit	Ratio of elastic limit to tensile strength	Elongation in 2 inches, per cent	Contraction of area, per cent
84,850	50,000	0.60	28.0	67.5
120,975	90,000	0.745	14.5	51.0
166,950	157,500	0.94	12.5	44.0
205,600	200,000	0.97	13.0	48.7
240,975	225,000	0.93	9.0	20.5

5. **Manganese Steel.**—Manganese, as we have seen, normally occurs in small quantities in most steels, promotes deoxidation and soundness, and makes sulphur practically harmless. If, however, when combined with high carbon, it is increased to from

<sup>1</sup> HIBBARD, p. 7.

5 to 10 per cent, it makes an alloy which has very little ductility, and is too hard and brittle (martensitic) for use; but if from 12 to 14 per cent, with carbon about 1 per cent, it produces an alloy which, with proper heat treatment (heating and quenching) combines ductility with great strength and toughness. This is much used to resist wear, and generally must be cast, as it is too hard to be rolled or machined. These facts show that if a certain percentage of an element produces certain effects, it does not follow that a larger percentage will produce the same effects in greater degree; it may produce opposite effects.

Manganese steel has not, in the knowledge of the writer, been used in structures. Its strength is much greater than that of structural steel (soft or medium carbon steel) but its elastic

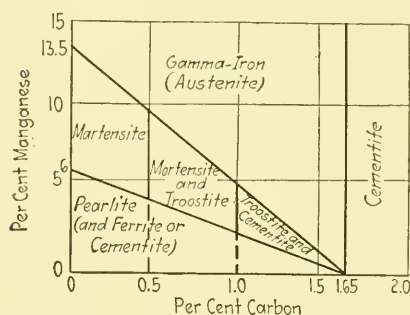


FIG. 35.—Constitution of manganese steels. (*Guillett Jour. Ir. and St. Inst.* vol. 70, p. 7.)

limit is lower in proportion to its ultimate strength. It could not be used for structural members; but it is used for parts where great hardness is desired, as for the jaws of rock crushers, and in some cases for railroad rails subject to specially heavy wear and for frogs and switches, and burglar-proof safes.

In the Boston subway cast manganese rails were placed in 1902 on a curve of 82 feet radius, to replace low-carbon steel rails; they lasted 14 years where the carbon steel rails had shown an average life of 44 days, with about the same wear. The manganese rails had 1.25 per cent *C* and 12 per cent *Mn*.

Manganese steel cannot be softened by heating followed by slow cooling. The constitution is shown by Fig. 35, which shows that without quenching it may be pearlitic, martensitic, or austenitic. It has the curious quality of stretching very uniformly throughout the length of a test piece. "In the pulling test the percentage of contraction of area is less than the elongation, a result directly opposite to that with simple as well as most alloy steels, in which the percentage of contraction is usually twice or more that of the elongation. The pulled test piece has a rather uniform stretch throughout its length, whereas carbon

steels, as is well known, have a largely increased amount of stretch near the point of fracture. When a piece of manganese steel is pulled, the increase of strength due to cold working (stretching) is greater than the decrease in cross-section due to contraction, so that a stretched part becomes stronger than the unstretched parts, and elongation then occurs in another place. As the pulling is continued, all parts of the pulled section stretch one after the other, with the result that when the piece is finally ruptured the stretch has been comparatively uniform. There is indeed an increased local extension and contraction close to the point of rupture, as with other ductile steels, but it is less marked in the manganese steels."<sup>1</sup>

Manganese steel is generally toughened by a heat treatment by heating it to about 1,050° C. and quenching in cold water.

Hibbard gives the following results of a test of cast, heat-treated manganese steel, forged down to the size given:

Diameter, 0.823 inch; length, 2 inches;

Per cent C, 1.10; Mn, 12.4; Si, 0.15; P, 0.06;

Tensile strength, 152,840; elastic limit, 56,400;

Per cent elongation, 51.0; contraction of area, 39.5.

The toughness, or resilience, or resistance to shock, may be roughly measured by the product of the ultimate strength and the elongation. This has been called the "merit number." In the case of the above steel it is 7,794,840. Hibbard states that the merit number of manganese steel (Mn 11 to 14 per cent) is "perhaps the greatest of all known steels." He gives the following table:

MERIT NUMBERS OF VARIOUS METALS

Metal	Tensile strength, pounds per square inch	Elongation, per cent	Merit number
Manganese steel.....	140,000	50.0	7,000,000
Soft steel.....	60,000	30.0	1,800,000
Tool steel.....	130,000	5.0	650,000
Cast iron.....	20,000	0.5	10,000
Nickel steel, natural.....	95,000	21.0	1,995,000
Nickel steel, heat-treated....	207,000	14.0	2,898,000

<sup>1</sup> HIBBARD.

Of course the accurate ultimate resilience depends upon the shape of the stress-strain diagram, as explained in Chap. IV of the previous volume.

The heat treatment described for manganese steel increased its tensile strength from about 88,000 to 145,000 pounds, its elongation from 3.5 to 50 per cent, and its merit number from 308,000 to 7,262,000, according to certain tests. The importance of heat treatment is obvious.

The relatively low elastic limit of manganese steel is noticeable. Remarkable, also, is the fact that sudden cooling, which produces brittleness in carbon steel, produces ductility in manganese steel.

Manganese confers a fine grained structure, even after slow cooling; and it shifts the eutectoid point toward the lower carbon content. One per cent of manganese is said to lower the eutectoid ratio to 0.78 per cent carbon.<sup>1</sup>

**6. Nickel Steel.**—Nickel steel is the most valuable alloy for the structural engineer, sometimes as a quaternary alloy with chromium (chrome nickel steel). Chromium is cheaper than nickel, and the chrome-nickel steels are increasing in favor.

Nickel does not occur in ordinary carbon steel, nor does it affect such steel in any beneficial way, as manganese and silicon do. It is added solely to confer desired qualities upon the alloy. These qualities depend not only on the amount of nickel, but also upon the carbon and the heat treatment. Simple nickel steel for structural purposes contains from 2 to 4 per cent nickel; generally about 3.25 is meant when nickel steel is mentioned without further qualifications. Such steel, with C. 0.25 and Mn about 0.86, will have, without heat treatment after rolling, a tensile strength of about 100,000 pounds and an elastic limit of nearly 70,000, with little reduction of ductility as compared with structural carbon steel. Perhaps its most valuable structural quality is that the elastic limit and the yield point are raised in greater proportion than the ultimate strength. In ordinary carbon steel the elastic limit should be at least half the ultimate strength, but in 3.5 per cent nickel steel it should be at least six-tenths. Thus, if the ultimate is increased 80 per cent, the elastic limit should be over twice that of structural carbon steel,<sup>2</sup> and

<sup>1</sup> *Scientific paper* 464. U. S. Bureau of Standards. See also *Scientific paper* 453.

<sup>2</sup>  $\frac{1.8 \times 0.6}{0.5} = 2.16.$



the allowable unit stress may be increased in about the same proportion. Nickel steel is thus peculiarly applicable in the case of long-span bridges, in which it is important to reduce the dead weight. It has been used in the Blackwell's Island, Hell Gate, and Manhattan bridges in New York, the Quebec bridge, the St. Louis Municipal bridge, and the Kansas City viaduct. The value of  $E$  for such steel is about the same as for carbon steel, about 29,500,000 pounds per square inch. With higher percent-

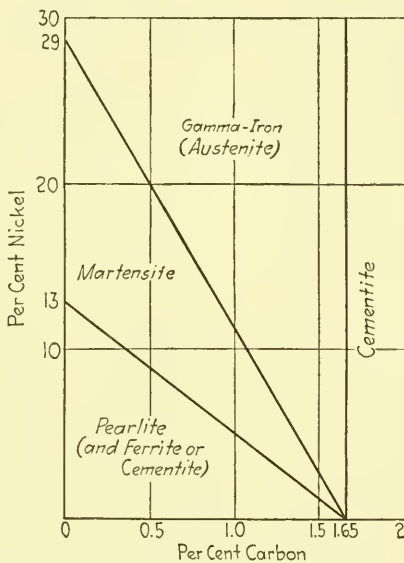


FIG. 36.—Constitution of nickel steels. (*Guillett Jour. Ir. and St. Inst.*, vol. 70 p. 4.)

ages of nickel, while the strength is greater, the modulus of elasticity may be lower, making the material less suitable for structures. Nickel steel is harder than carbon steel, corrodes less, and has a lower melting point, so that it is very suitable for castings. On the other hand, nickel hinders the welding of steel, so that if there are blowholes they are less apt to be welded up in rolling, but are more likely to be drawn out into injurious seams; but if oxidized they will probably be so drawn out in any steel.

Guillet's diagram of the constitution of nickel steel, not heat treated, is shown in Fig. 36. Pearlite is strong and ductile; martensite stronger and more brittle; austenite stronger than

pearlite but less strong than martensite, and at the same time tough and ductile. Hence it would be expected that low nickel steels (pearlitic) and high nickel steels (austenitic) would be the most useful except where great hardness is desired; and since nickel is expensive, low nickel steels are those most used. The modulus of elasticity of high nickel steel is lower than that of low nickel steel (only about 23,000,000 pounds per square inch), hence the former yield more under stress, and are less suitable for structures even if the cost were not high.

Nickel steel with 22 per cent nickel has been used for the valve stems of the salt-water fire protection service of New York, where it was essential that corrosion should be prevented. Generally speaking, the high nickel steels are only used where resistance to corrosion is very important (as in boiler tubes); or for their high electrical resistance.

Owing to the lowering of the transformation temperature, nickel steel may therefore be pearlitic, martensitic, or austenitic, without quenching, depending upon the percentages of nickel and carbon.

Alloys with nickel 10 to 25 per cent, are martensitic, and by quenching become austenitic, that is, are softened. With nickel about 20 per cent, the transformation temperature from the austenitic condition, on cooling, is lowered to below  $100^{\circ}\text{C.}$ ; and, on reheating, the austenitic condition will not be resumed until a temperature of about  $600^{\circ}\text{C.}$  is reached, whereas in ordinary carbon steel the heating transformation point is only 25 to  $50^{\circ}$  above the cooling point. This means that the alloy may exist between 100 and  $600^{\circ}$  in either condition—martensitic or austenitic. The two ends of the same bar may thus be made to possess, within those temperatures, quite different properties, one end being magnetic and the other not. With 20 per cent nickel, 1 per cent carbon, and 1.4 per cent manganese, the transformation temperature on cooling is reduced to about  $188^{\circ}\text{C.}$  below zero; and on heating to above usual atmospheric temperature. With over 25 per cent nickel, the heating and cooling transformation temperatures are about the same. This phenomenon is expressed by the statement that nickel steels with less than 25 per cent nickel are irreversible, while those with more are reversible.

“Invar” steel, containing 36 per cent nickel, has the peculiarity that it has a very low coefficient of expansion. It is used for steel tapes, clock pendulums, etc.

With 42 per cent nickel, the metal, known as "platinite," has the same coefficient of expansion as glass, and can therefore be used as wire in "wire glass" and in electrical connections through glass without cracking the glass under temperature changes.

Nickel steel resists shock and fatigue better than carbon steel. It will no doubt be used more and more in structures.

**7. Chrome Steel.**—Chrome steel, which is practically always heat-treated by quenching and drawing, is stronger and much harder than carbon steel with the same carbon content, and is also ductile and tough. The percentage of chromium does not often exceed 2 per cent and is generally 1 per cent or less. The elastic limit is nearly 90 per cent of the ultimate and sometimes more. Chrome steel is used in mining machinery, and for vaults, rolls, files, balls and rollers for bearings, armor plate, projectiles, and certain automobile parts which require hardness. It is largely giving way to chrome-nickel steel.

**8. Silicon Steel.**—Silicon, as we have seen, is frequently added to steels to promote soundness by absorption of oxygen. The percentage in ordinary steel is usually below 0.3. If the percentage is increased to from 0.3 to 0.5, a considerable increase of strength is obtained, with little loss of ductility. Silicon steel with not over 0.45 has been used in the new Delaware River bridge towers. This is a suspension bridge, the cables being of 0.85 carbon steel with an ultimate strength of 215,000 and a yield point of 144,000 pounds per square inch. The specifications for the towers are given at the end of this chapter.

For electrical purposes, a silicon steel developed by Hadfield has been used, which has very high magnetic permeability and electrical resistance, and low hysteresis. It contains 4 to 4.5 per cent of silicon, and the smallest possible amount of carbon, manganese, and other impurities, and is subjected to heat treatment by heating first to 1,070° C., cooling quickly to atmospheric temperature, then heating to 750° C. and cooling slowly, afterwards sometimes again heating to 800° C. and cooling slowly.<sup>1</sup>

Silicon steel is also used for automobile springs, and will probably be more extensively used in structures.

**9. Copper Steel.**—In Chap. XIII, reference is made to the use of steel with about 0.25 per cent copper, to resist atmospheric corrosion.

<sup>1</sup> STOUGHTON, p. 444.

**10. Vanadium** in very small percentages (less than 0.2) and with suitable heat treatment, increases greatly the strength of carbon steel, especially against shock and alternating stresses. It is being more and more used, especially with chromium. Vanadium is a deoxidizer, and therefore promotes soundness. Chrome-vanadium steel is used extensively for automobile parts. It is more free from seams than alloys with nickel.

**11. Chrome-nickel** steels are important structural materials. By suitable proportioning and heat treatment (they are practically always heat-treated) they "can be made to have as high physical properties as any steels known, with any elastic limit between 40,000 and 250,000 pounds per square inch, accompanied by ductility that is high as compared with its strength." As chromium is cheaper than nickel, chrome-nickel steels can be made more cheaply than simple nickel steel of the same strength and ductility.<sup>1</sup> These steels are used in automobiles, and for armor plate and projectiles.

Mayari steel is a chrome-nickel steel from ore mined at Mayari in Cuba. It was first used in bridge construction in the Memphis bridge, by Ralph Modjeski, and has since been used in a number of bridges. It contains from 1 to 1.5 per cent nickel and 0.20 to 0.75 per cent chromium.

*Chrome-vanadium* steels are much used for automobile parts, and for some other purposes. They are much like chrome nickel steels, but lack the imperfection which nickel steels sometimes possess, namely the tendency to have seams if the ingots have blowholes. This is because vanadium is a deoxidizer, while nickel is not.

**12. High-speed Tool Steels, or "Rapid" Steels.**—With 16 to 20 per cent tungsten, 2 to 6 per cent chromium, carbon 0.5 to 0.7 and a small amount of vanadium, a steel is produced which, with suitable heat treatment, will retain great hardness when red-hot. This alloy, and the method of heat treatment, were discovered by F. W. Taylor<sup>2</sup> and Maunsel White, of the Bethlehem steel works; and, used as a cutting tool it has revolutionized the art of cutting metals, making it possible to take heavier and more rapid cuts than formerly. Tungsten is considered to be the element that gives the steel hardness and toughness at a red heat, and its use makes the best percentage of carbon only about half that

<sup>1</sup> HIBBARD.

<sup>2</sup> TAYLOR, F. W.: The Art of Cutting Metals; *Trans. A.S.M.E.*, 1906.

required in cutting tools of carbon steel. The heat treatment consists in heating nearly to the fusion point, or line *AD* in Fig. 11 and quenching in oil, with sometimes a second heating. No tempering change occurs on reheating to below 550° C., indicating a stable condition. The action of these steels is said to be due to the formation of a double carbide of chromium and tungsten.

13. A steel is called "self-hardening" or "air-hardening" when it is hard and austenitic or martensitic when cooled slowly, that is, when the transformation point has been lowered to below atmospheric temperature. Self-hardening steels which remain austenitic are non-magnetic because the ferrite is in the  $\gamma$ -form. Those which become martensitic, which is more usual, are magnetic.

The action of alloying elements, even in small amounts, is often very peculiar. An alloy steel with 9 per cent tungsten, 2.5 per cent manganese, and 1.85 per cent carbon is incapable of being made soft by any known process. The same result is produced if the manganese is replaced by 1 or 2 per cent chromium. This is peculiar, because tungsten alone does not reduce the critical temperature, while chromium alone raises it slightly, though the combination reduces it below atmospheric temperature.

14. There follow extracts from the specifications for the new Quebec bridge (1913), and for the main towers of the Delaware River bridge (1922).

The structural engineer will have little occasion to deal with alloy steels unless he has to build a very large bridge. In that case he should investigate the subject and obtain the counsel of a competent metallurgist. The subject is a specialized and intricate one. Up to the present time nickel steel, chrome-nickel steel, and silicon steel are the only alloys much used in structures.

The mechanical engineer, especially if concerned with automobiles, will find it necessary to inform himself on the subject.



## SPECIFICATIONS FOR THE NEW QUEBEC BRIDGE (1913)

## Rolled Carbon Steel

162. *Furnace Used.*—All structural steel shall be made in an open-hearth furnace.

166. *Chemical Requirements.*—The ladle tests of steel as usually taken shall not contain more than the following proportions of the elements named:

	ACID, PER CENT	BASIC, PER CENT
Phosphorus.....	0.06	0.04
Sulphur.....	0.04	0.04
Manganese.....	0.70	0.70
Except rivet steel.....	....	0.60
No chromium to be used.		
Silicon.....	0.10	0.10

It is desired that the carbon contents be as small as possible to meet the specifications.

167. *Rivet Steel.*—The ladle tests of the carbon rivet steel shall not contain more than 0.03 of 1 per cent of phosphorus, and not more than 0.03 of 1 per cent of sulphur.

170. *Physical Requirements.*—Specimens cut from the finished material shall show the following physical properties:

Material	Ultimate strength, pounds per square inch	Minimum yield point, pounds per square inch	Minimum elongation, per cent in 8 inches	Minimum reduction, per cent of area
Shapes and plates up to and including 1 inch thick....	62,000 to 70,000	35,000	<u>1,500,000</u> ultimate	44
Shapes and plates over 1 inch thick.....	62,000 to 70,000	33,000	22 and 20 per cent for sheared plates	40
Eye-bar flats (unannealed)...	66,000 to 74,000	35,000	22 per cent	40
Rivets.....	48,000 to 56,000	28,000	1,500,000 ultimate	50
Pins and rollers (annealed)...	65,000 to 75,000	35,000	22 per cent in 2 inches.	35

Yield to be determined by drop of the beam.

Speed of machine for testing samples to be such that material under tension will not elongate more than 1 inch in two minutes.

171. *Bending Tests*.—Specimens cut from plates, bars and shapes 2 inches wide shall bend cold  $180^{\circ}$  around a rod of a diameter equal to the thickness of the specimen; when at or above a red heat,  $180^{\circ}$  flat.

Specimens cut from rivet rods shall bend  $180^{\circ}$  flat when cold, or when at or above red heat. A test piece 2 inches long when heated to a bright cherry red shall flatten longitudinally under the hammer to a thickness of one-quarter ( $\frac{1}{4}$ ) inches without cracking on the edges.

Full-sized sections of eye-bar material as rolled without annealing shall bend cold about a rod of diameter equal to twice the thickness of the bar. Angles of all thicknesses shall open cold to an included angle of  $150^{\circ}$  and close to an angle of  $30^{\circ}$  without a sign of fracture.

All specimens in bending tests must show no signs of cracking on the outside of the bend.

172. *Fractures in Tension*.—The fracture of all tension tests shall show a fine silky texture, of a uniform bluish grey or dove colour, free from black or brilliant specks, and show no sign of crystallization.

### Rolled Nickel Steel

173. *Furnace*.—All nickel steel shall be made in an open-hearth furnace. It shall be made in the same manner as specified for rolled carbon steel, with the addition of nickel.

174. *Chemical Requirements*.—The ladle test shall contain not less than 3.25 per cent of pure nickel, and not more than the following proportions of the elements named:

	ACID, PER CENT	BASIC, PER CENT
Phosphorus.....	0.06	0.04
Sulphur.....	0.04	0.04
Manganese.....	0.70	0.70
No chromium to be used.		
Silicon.....	0.10	0.10
Carbon.....	0.45	0.45

175. *Heating and Rolling*.—Care shall be taken in the heating and rolling of nickel steel to prevent the formation of heavy scale. The material must not be pitted by rolling the scale into it. All material with pitted or heavily scaled surfaces, or with ragged edges, will be rejected.

176. *Physical Requirements*.—Nickel steel for plates, shapes, and unannealed eye-bar flats must meet the following physical requirements in the finished material:

Material	Ultimate strength, pounds per square inch	Minimum yield point, pounds per square inch	Minimum elongation, per cent in 8 inches	Minimum reduction, per cent of area
Plates and shapes. . . .	85,000–100,000	50,000	$\frac{1,600,000^1}{\text{ultimate}}$	40 <sup>3</sup>
Eye-bar flats, un-annealed <sup>2</sup> . . . . .	95,000–110,000	55,000	15 per cent	25
Pins, annealed. . . . .	90,000–105,000	55,000	$\frac{1,800,000}{\text{ultimate}}$ in 2 inches	35

<sup>1</sup> For material thicker than one inch (1 in.), the required percentage of elongation shall be reduced by one for each increase in thickness of one quarter inch ( $\frac{1}{4}$  in.) or fraction thereof above one inch (1 in.), but in no case shall the minimum elongation required be less than 14 per cent.

<sup>2</sup> Tests for information shall be made of annealed specimens cut from the rolled eye-bar flats.

<sup>3</sup> For material thicker than three-quarter inch ( $\frac{3}{4}$  in.) the required percentage of reduction of area shall be reduced by two for each increase in thickness of one-quarter inch ( $\frac{1}{4}$  in.) or fraction thereof above three-quarter inch ( $\frac{3}{4}$  in.)

Bronze bushings used in heads of hanger eyebars for suspending center span and bearing plates with sliding surfaces must meet the following physical requirements:

In tension	POUNDS PER SQUARE INCH
Tensile strength. . . . .	100,000
Elastic limit. . . . .	60,000
Elongation in 2 inches. . . . .	10 %
Reduction of area. . . . .	20 %

In compression

Elastic limit. . . . . 60,000

Permanent set due to a load of 100,000 pounds per square inch, 0.02.

177. *Bending Tests.*—Specimens of nickel steel not less than 2 inches wide and of the full thickness of the material as rolled shall bend cold 180° around rods of the diameters specified below for the various thicknesses, without fracture on the outside of the bend.

For material up to  $\frac{1}{2}$  inch, inclusive. . . . . 180° around  $D = 1T$

For material over  $\frac{1}{2}$  inch and up to  $1\frac{1}{2}$  inches,  
inclusive. . . . . 180° around  $D = 2T$

For material over  $1\frac{1}{2}$  inches. . . . . 180° around  $D = 3T$

Angles of all thicknesses shall open cold to an included angle of 150° and close to an angle of 30°, without a sign of fracture.

### Steel Castings

182. *Chemical Requirements.*—The ladle test of steel for castings shall not contain more than the following proportions of the elements named:

Phosphorus.....	0.04 of 1 per cent for basic steel.
Phosphorus.....	0.06 of 1 per cent for acid steel.
Sulphur.....	0.05 of 1 per cent.
Manganese.....	0.75 of 1 per cent.
Silicon.....	0.35 of 1 per cent.

183. *Annealing.*—All steel castings shall be carefully and thoroughly annealed in a manner approved by the engineer, and shall have a fine-grained or silky fracture.

### FROM SPECIFICATIONS FOR THE MATERIAL FOR TWO MAIN TOWERS OF THE DELAWARE RIVER BRIDGE (1922)

All structural steel shall be made by the open-hearth process.

Two kinds of wrought structural steel will be used in the towers, designated respectively as carbon steel and silicon steel.

The various grades of steel shall not contain more than the following percentages of elements:

	Carbon steel			Silicon steel
	Mild	Medium	Rivet	
Carbon.....	....	....	.....	0.40
Manganese.....	....	....	.....	1.00
Phosphorus:				
Acid process.....	0.06	0.06	0.04	0.06
Basic process.....	0.04	0.04	0.04	0.04
Sulphur.....	0.05	0.05	0.045	0.05
Silicon.....	....	....	.....	0.45

The percentage of silicon in silicon steel shall not be less than 0.20. Specimens cut from the finished material shall show the following physical properties:

	Carbon steel			Silicon steel
	Mild	Medium	Rivet	
Tensile strength, pounds per square inch.....	55,000-65,000	62,000-70,000	52,000-60,000	80,000-95,000
Minimum yield point, pounds per square inch	$\frac{1}{2}$ tensile strength	37,000	30,000	45,000
Elongation in 8 inches, minimum per cent....	1,400,000 <sup>1</sup>	1,500,000 <sup>1</sup>	1,500,000	1,600,000 <sup>2</sup>
	tensile strength	tensile strength	tensile strength	tensile strength
Reduction of area, minimum per cent		42 per cent <sup>3</sup>	52 per cent	35 per cent <sup>4</sup>
Bend test, mat'l. $\frac{3}{4}$ inch or less.....	180° flat	180° around $D = T$	180° flat	180° around $D = T$
mat'l. over $\frac{3}{4}$ to $1\frac{1}{4}$ inches.....	180° around $D = T$	180° around $D = 1.5T$		180° around $D = 1.5T$

$D$  = Inside diameter of bend.

$T$  = Thickness of material.

<sup>1</sup> For mild or medium steel material over  $\frac{3}{4}$  inch thick, deduct one from percentage of elongation for each increase in thickness of  $\frac{1}{8}$  inch or fraction thereof, above  $\frac{3}{4}$  inch, but in no case shall the elongation be less than 18 per cent.

<sup>2</sup> For silicon steel material over 1 inch thick, deduct one from percentage of elongation for each increase in thickness of  $\frac{1}{4}$  inch or fraction thereof, above 1 inch, but in no case shall the elongation be less than 14 per cent.

<sup>3</sup> For medium steel material over  $\frac{3}{4}$  inch thick, deduct one from percentage of reduction of area, for each increase in thickness of  $\frac{1}{8}$  inch, or fraction thereof, above  $\frac{3}{4}$  inch, but in no case shall the reduction of area be less than 35 per cent.

<sup>4</sup> For silicon steel material over  $\frac{3}{4}$  inch thick, deduct one from percentage of reduction of area for each increase in thickness of  $\frac{1}{8}$  inch, or fraction thereof, above  $\frac{3}{4}$  inch, but in no case shall the reduction of area be less than 24 per cent.

12. Tests from material of a thickness or diameter in excess of  $1\frac{1}{2}$  inches, shall show an ultimate strength and yield point equal to the minimum specified for its grade and an elongation in 2 inches of 1,600,000 tensile strength.

13. All angles shall withstand being opened when tested cold, to an angle of 150°, or closed to an angle of 30°, without rupture.

14. All silicon steel shall be made especially for this work and shall be subject to a system of identification approved by the Engineer and shall be handled by itself and isolated in such manner as to prevent the possibility of its becoming mixed with other kinds of steel.

15. All steel shall be of uniform quality of each class. It shall be straight, without buckles or kinks, and free from injurious seams, flaws, cracks, excessive scale and pitting and other defects.

16. Every finished piece of structural steel shall have the melt number and name of the manufacturer stamped or rolled on it. Shapes and universal mill plates shall be hot stamped. Sheared plates shall be stamped



with dies after laying out. Painting melt numbers will not be sufficient. Rivet steel and similar small pieces may be shipped in bundles, securely tied together, with melt number attached.

17. Whenever any steel is to be allowed to become cold in any stage of rolling, every individual piece shall be distinctively stamped with its melt number while still hot.

18. All cold ingots, blooms or slabs, before being heated preliminary to further working, shall have their heat numbers identified and approved by the inspector.

19. The cross-section or weight of each piece of wrought steel shall not vary more than  $2\frac{1}{2}$  per cent from that specified, except in the case of sheared plates, for which allowance will be made in accordance with the specification of structural steel for bridges of the American Society for Testing Materials (A7-21), a copy of which is on file in the office of the Chief Engineer.

29. The yield point shall be determined by the drop of the beam of the testing machine. The testing machine shall not be stopped to obtain the drop of the beam.

30. All tension fractures shall be silky and of fine texture, free from coarse crystals. Square fractures shall be a sufficient cause for rejection.

31. In case the ultimate strength falls outside of the specified limits by less than 1,000 pounds, all other requirements being filled, or in case the yield point falls below the specified minimum by less than 1,000 pounds, all other requirements being filled, then two more tests may be taken from material of the same melt and thickness for each test thus failing and if both such retests fill all requirements the material will be accepted.

32. The following table is reproduced, by permission, from the valuable book by Professor H. F. Moore, on the "Materials of Engineering," McGraw-Hill Book Co., Inc., 1922, and gives a summary of average values compiled by him.

## AVERAGE VALUES FOR STRENGTH, STIFFNESS, AND DUCTILITY OF IRON AND STEEL

The values given in this table are based on test data from various materials testing laboratories

Material	Strength in tension, pounds per square inch		Strength in compression, pounds per square inch		Strength in shear, pounds per square inch		Modulus of elasticity, pounds per square inch		Elongation in 2 inches, per cent
	Proportional limit	Ultimate	Proportional <sup>1</sup> limit	Ultimate	Proportional limit	Ultimate	Tension	Shear	
Gray cast iron.....	<sup>1</sup>	20,000	26,000	75,000	<sup>2</sup>	<sup>3</sup>	15,000,000	6,000,000	Slight
Malleable cast iron.....	15,000	45,000	15,000	<sup>4</sup>	10,000		23,000,000	10,500,000	7.5
Electrolytic iron.....	20,000	40,000	20,000	<sup>4</sup>	12,000	30,000	27,000,000	10,000,000	50.0
Commercial wrought iron.....	30,000	50,000	30,000	<sup>4</sup>	18,000	35,000	27,000,000	10,000,000	35.0
Steel, 0.10 per cent carbon:									
Rolled metal.....	30,000	50,000	30,000	<sup>4</sup>	18,000	35,000	30,000,000	12,000,000	45.0
Steel, 0.20 per cent carbon:									
Rolled metal.....	35,000	60,000	35,000	<sup>4</sup>	21,000	45,000	30,000,000	12,000,000	35.0
Castings.....	30,000	60,000	30,000	<sup>4</sup>	18,000	45,000	30,000,000	12,000,000	30.0
Steel, 0.40 per cent carbon:									
Rolled metal.....	50,000	90,000	50,000	<sup>4</sup>	30,000	65,000	30,000,000	12,000,000	20.0
Castings.....	40,000	80,000	40,000	<sup>4</sup>	24,000	60,000	30,000,000	12,000,000	15.0
Steel, 0.60 per cent carbon:									
Rolled metal.....	60,000	115,000	60,000	<sup>4</sup>	36,000	85,000	30,000,000	12,000,000	14.0
Steel, 0.80 per cent carbon:									
Rolled metal.....	70,000	135,000	70,000	<sup>4</sup>	42,000	100,000	30,000,000	12,000,000	10.0
Steel, 1.00 per cent carbon:									
Annealed rolled metal.....	80,000	150,000	80,000	<sup>4</sup>	48,000	115,000	30,000,000	12,000,000	5.0
Heat-treated rolled metal.....	160,000	200,000	160,000	<sup>4</sup>	96,000	150,000	30,000,000	12,000,000	Slight
Cold-rolled steel, 0.20 carbon.....	60,000	80,000	60,000	<sup>4</sup>	36,000	60,000	30,000,000	12,000,000	18.0



## CHAPTER IX

### NONFERROUS METALS AND ALLOYS

#### REFERENCES:

MILLS, Sec. 6; JOHNSON, Chap. XXVI; UPTON, Chap. XIX.

1. The structural engineer has little or nothing to do with these materials. Some of the brasses (copper-zinc alloys) and some of the bronzes (copper-tin alloys) are used in machinery, and are of great importance there. Copper, and sometimes aluminum, are used for wires carrying electric current, and in the design of transmission towers may have to be considered as structural materials.

Only a few brief notes are thus required here regarding these materials, and the reader is referred to the following works for detailed information (see Chap. I).

2. **Copper** is a very ductile and malleable metal with high electrical conductivity. Its production is one of the great industries of the United States, which country produces more than one-half of the world's production.

The strength of copper depends largely upon the mechanical treatment to which it has been subjected. Hot-rolled copper plate has an elastic limit of only 7,000 or 8,000 pounds per square inch, an ultimate tensile strength of about 33,000, a percentage of elongation of about 50: by cold hammering, the elastic limit may be raised to above 20,000, the percentage of elongation reduced to 30, the ultimate strength remaining unchanged. Copper wire, according to the specifications of the A.S.T.M., has properties as shown by the following table:

COPPER WIRE

(A.S.T.M.)

Diameter, inches	Hard-drawn		Medium hard-drawn			Soft wire	
	Minimum tensile strength, pounds per square inch	Minimum per cent of elonga- tion in 10 inches	Tensile strength		Minimum per cent of elonga- tion in 10 inches	Minimum tensile strength, pounds per square inch	Minimum per cent of elonga- tion in 10 inches
			Minimum, pounds per square inch	Maximum, pounds per square inch			
0.460	49,000	3.75	42,000	49,000	3.75	36,000	35
0.325	54,500	2.40	45,000	52,000	3.00	36,000	35
0.229	59,000	1.79	48,000	55,000	2.25	37,000	30
.....	.....	per cent in 60 inches	.....	.....	per cent in 60 inches	.....	.....
0.204	60,100	1.24	48,330	55,330	1.25	37,000	30
0.102	64,900	1.00	50,330	57,330	1.04	38,500	25
0.040	67,000	0.85	53,000	60,000	0.88	38,500	25
0.003	.....	.....	.....	.....	.....	40,000	20

These figures show the increase of strength and the decrease of ductility as the diameter decreases, that is, as the mechanical work put upon the metal increases. The effect of partial annealing is shown in the figures for medium hard-drawn wire, and that of nearly complete annealing in those for soft wire. The elastic limit of hard-drawn wire will be 55 to 60 per cent of the ultimate, and about 50 per cent for the medium hard-drawn wire. The yield point is not well defined. The modulus of elasticity is about 16,000,000, varying from about 11,000,000 for cast copper to 17,600,000 for hard-drawn copper wire. Copper resists corrosion much better than iron or steel, and is sometimes used as a roof covering, and also for water conductors, wires for screens, etc. The shearing strength of cast copper is about 30,000 pounds per square inch.

**3. Zinc** is not used in structural work, except as a thin coating for iron or steel (galvanizing). The properties of cast zinc, all having small (varying) proportions of impurities, are about as follows, the figures being averages:<sup>1</sup>

Tensile strength: 3,700 to 11,980 pounds per square inch.

Compressive strength at 20 per cent deformation in 2.6 inches:

23,030 to 39,490 pounds per square inch.

Modulus of rupture: 10,050 to 16,550 pounds per square inch.

Shearing strength: about 17,000 pounds per square inch.

Modulus of elasticity: 10,000,000 to 15,000,000 pounds per square inch.

<sup>1</sup> See RIGG and WILLIAMS, *Proc. A.S.T.M.*, vol. 13, p. 669.



Some of the variations are due to the impurities, others to the physical structure. As in the case of copper, the strength of zinc is affected by the mechanical treatment. Zinc is less ductile than tin.

**4. Tin** is one of the few metals which are not found in large quantities in the United States. It is mainly found in Great Britain, the East Indies, Australia, and South America. It has two allotropic forms, white tin and gray tin. The former is the usual form, is very malleable, very resistant to corrosion, and has small tensile strength, being but little stronger than lead, or from 3,000 to 5,000 pounds per square inch. It is used for tin-foil; and as sheet tin is used for cans and for roofing. Tin plate is sheet iron coated with tin. Gray tin is a dust or powder, and white tin when heated moderately tends to change to the gray form and become useless, especially when hammered. The formation of gray tin is called the "tin disease." Large quantities of tin are used, with copper, to form *bronzes*, which are very useful, and possess different properties from either constituent.

**5. Lead.**—Lead is easily fusible and is a plastic material, having only minor uses in structural engineering; sometimes under bed plates to give an even bearing on a rough surface, sometimes poured in liquid form as a filling around bolts set into stone.

**6. Aluminum** is used widely in the form of tubing and for many domestic articles, as well as for wire. It is ductile, very malleable, much hardened by cold working, and very light, but finds little application in structures (see Art. 9). Its tensile strength varies from about 11,000 pounds per square inch cast to 55,000 pounds as hard-drawn wire. Its compressive strength is about 67,000 pounds per square inch. It is sometimes used alloyed with copper for parts of automobiles and aeroplanes.

**7. Nickel** is a silvery metal, very resistant to corrosion, and with a tensile strength ranging from 75,000 pounds per square inch to double that value as hard-drawn wire. In structures its principal use is to form an alloy with steel, a small proportion of nickel increasing the strength and elastic limit of the steel. Almost all the nickel used comes from the Sudbury district of Ontario, Canada, and is produced by the International Nickel Company, although a small amount comes from New Caledonia.

**8.** The *brasses*, which are alloys of copper and zinc, and the *bronzes*, which are alloys of copper and tin, are the most important nonferrous alloys. There are also alloys of copper, zinc, and tin.

These alloys were exhaustively treated by R. H. Thurston, in the third volume of his work on the "Materials of Engineering." See also the report of the U. S. Board to test iron, steel, and other metals, 1881.

The addition of zinc increases the strength up to about 45 per cent zinc, and the ductility up to about 30 per cent zinc. With further additions of zinc, these qualities rapidly diminish, and when the percentage of zinc reaches 50 or 60, the alloys are so weak and brittle as to be practically worthless. There are many brasses. The most usual composition is about two-thirds copper and one-third zinc. "Muntz" metal contains 60 per cent copper and 40 per cent zinc, and is used for bolts, nuts and other purposes. "Manganese bronze" (really a brass) contains copper 53 to 62, zinc 36 to 45, aluminum 0.05 to 0.5, and lead not over 0.15, with small amounts of tin and manganese, and has a tensile strength, as cast, of above 70,000 pounds per square inch (A.S.T.M. specifications). It is called manganese bronze because a small quantity of manganese is added to the melted mass, during manufacture, for its deoxidizing effect, although the resultant alloy contains little or no manganese. It is very resistant to corrosion and to wear (like phosphor bronze).

The tensile strength of brass reaches 45,000 to 50,000 pounds per square inch of original section when the composition is about 55 to 60 copper and 40 to 45 zinc; and with increasing zinc it decreases to about 5,000 for cast zinc. The modulus of elasticity of rolled brass is about twelve to thirteen million pounds per square inch. Brass is used for hardware, wire, tubes, valves, and fittings, and many other purposes, particularly where a non-corrosible material is necessary.

9. The addition of tin to copper, forming bronze, increases the hardness and tensile strength up to about 20 per cent tin, and the compressive strength up to about 25 per cent, after which the strength falls off rapidly. The highest compressive strength is about 150,000 pounds per square inch, and the highest tensile strength about 35,000 pounds per square inch. The ductility, however, falls off rapidly above about 5 per cent tin. The properties are considerably affected by heat treatment. The modulus of elasticity is between ten and twelve million pounds per square inch. The best brasses exceed the best bronzes in tensile strength and ductility, but seem inferior in compressive strength. One of the strongest bronzes is *gun metal*, containing

about 90 per cent copper and 10 per cent tin. *Bell metal* is used for bells, on account of its resonance, and contains 20 to 25 per cent tin, but is brittle. *Machinery bronzes* are used for bearings, valves, and other machinery parts, and contain 81 to 87 per cent copper, many being ternary alloys of copper, tin, and zinc, with other elements in some cases.

Bronze bearing metal for turntables and movable railroad bridges consists of copper and tin, with 11 to 20 per cent tin, not over about 1 per cent phosphorus, and nearly all the remainder copper.

*Phosphor-bronze* is bronze which has been deoxidized by phosphorus, of which the product may contain very little, and which contains 87 to 92 per cent copper, 8 to 10 per cent tin, and up to 2.5 per cent zinc. It is used for engine parts, pinions, propeller screws, hydraulic cylinders, valve rods, and other parts, especially where it is important to resist corrosion, to which it is remarkably resistant. It is very hard and very resistant to wear.

Aluminum and silicon also are deoxidizers, and are used in silicon-bronze and aluminum-bronze. The latter contains no tin, and, therefore, is not properly a bronze, but is 90 to 95 per cent copper and 5 to 10 per cent aluminum. Its strength reaches 60,000 to 74,000 pounds per square inch with about 10 per cent of aluminum.

The hydraulic engineer is concerned with many metal parts which carry a load and are also exposed to friction, such as valves, and stems or rods for sluice gates or valves, which are operated by screws. In order to make such gates as easy to lift as possible, it is desirable to make the diameter of the screws small, so as to reduce the frictional moment. The smaller the screw, the smaller the frictional moment and the more easily the gate can be lifted, if the coefficient of friction remains the same. Such parts, too, are often idle for considerable periods, and thus are liable to corrode, with resulting increase of the friction. Hence the desirability of using a material which does not corrode.

An interesting and light alloy is *duralumin*, which has copper 4.5 per cent, aluminum 94.4 per cent, magnesium 0.6 per cent and manganese 0.5 per cent. It has, after rolling, a tensile strength of 30,000 to 35,000 pounds per square inch, but by heat treatment the tensile strength may be raised to 50,000 to 60,000 pounds per square inch, so that it combines great lightness with strength, and has been used in the construction of Zeppelins and aeroplanes.

Other light aluminum and copper alloys have been used in automobiles, such as one containing 92 per cent aluminum and 8 per cent copper, which has a tensile strength of 20,000 to 24,000 pounds per square inch, an elastic limit of 13,000, and weighs only about 175 pounds per cubic foot, as against 490 for steel.<sup>1</sup>

10. Ternary alloys, of copper, zinc, and tin are numerous, and may be called brasses or bronzes, according as zinc or tin predominates; or they may be termed *alloy brasses or bronzes* (U. S. Bureau of Standards).

Professor R. H. Thurston studied the strength of these alloys, and proposed a method of representing the results that has been

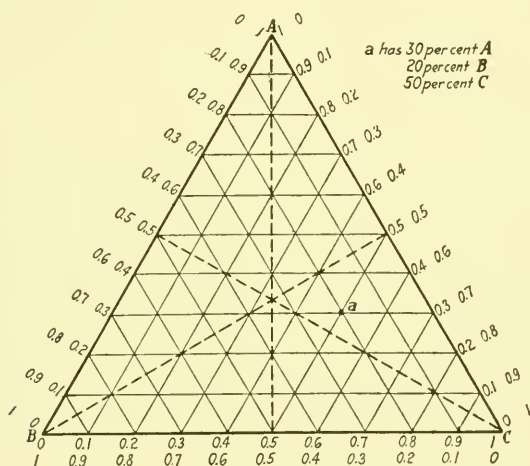


FIG. 37.

widely used in cases where there are three variables.<sup>2</sup> It is founded on the principle that in an equilateral triangle  $ABC$ , (Fig. 37) the sum of the perpendicular distances from any point to the three sides is constant. Thus if  $A$ ,  $B$  and  $C$  represent 100 per cent of three variable elements respectively, any point in the triangle will have a percentage of  $A$  proportional to its perpendicular distance from  $BC$ . Thus, dividing each side into tenths and drawing lines as shown, point  $a$  has 0.30 $A$ , 0.20 $B$ , and 0.50 $C$ . Lines drawn on the triangle through points for which the strength

<sup>1</sup> See *Cir. No. 76*, U. S. Bureau of Standards, on "Aluminum and Its Light Alloys."

<sup>2</sup> "The Strongest of the Bronzes," *Trans. A.S.C.E.*, vol. 10, pp. 1-13, 1881.

is the same will be contour lines of equal strength. Any function studied, such as strength, elongation and yield point, may be conceived represented by a surface formed by erecting at each point of the triangle a line proportional to the value of the function when the composition corresponds to the point. This method of representation should be fully understood by the reader. It is useful in many cases, such as in studying the effects, in concrete, of different proportions of fine, coarse, and medium sand.

Professor Thurston found the strongest of the bronzes to correspond nearly to copper 55, zinc 43, tin 2, parts in a hundred. It had a tensile strength of about 68,900 pounds per square inch of original area, or over 92,000 pounds per square inch of fractured area, an elongation of 47 to 51 per cent (length not stated) and a reduction of area of about 30 per cent.

*Tobin bronze*, which was discovered by J. A. Tobin, of the U. S. Naval Engineer Corps, about or before 1875, has copper 58.22, zinc 39.48, tin 2.30, as stated by Thurston, and, as cast, has a tensile strength of 66,500 pounds per square inch of original section, and 71,378 pounds per square inch of fractured section. Rolled hot, its tenacity rose to 79,000, and rolled cold to 104,000 pounds per square inch. Its elastic limit is about 60 per cent of the ultimate, and its elongation 15 to 25 per cent, or more, in 8 inches. It thus has the strength of steel, and is non-corrodible. It can be forged or rolled at a low red heat or worked cold, and can be bent double either hot or cold.<sup>1</sup> It sometimes contains small quantities of lead and iron.

**11. Bearing Metals.**—The mechanical engineer is concerned with metals to be used in bearings of machinery, such as journal bearings. Here the strength and hardness must be sufficient to carry the load, and the coefficient of friction should be small. A metal which is too hard will be apt to have too large a coefficient of friction, and one that is too soft will not have the proper resistance to wear. If the surface is composed of hard and soft particles, the former will support the load and resist the wear, while the latter will be worn down slightly below the hard particles, and so will afford opportunity to retain the oil used as a lubricant; also allowing the hard particles to be worn to a perfectly smooth surface.

<sup>1</sup> THURSTON.



A common bearing material is *Babbitt metal*, which consists of tin, copper, antimony and lead. There are many kinds of bearing metals on the market, some hard and some soft, for high and low pressures respectively. For turntables and movable bridges, where the speed of motion is slow and the pressure large, the metal used is a bronze, which has been referred to in Art. 9. For car journals a softer metal is preferable; such as one having about 77 parts copper, 8 tin, and 15 lead.

12. The weights of the nonferrous metals are about as follows:

METAL	WEIGHT PER CUBIC FOOT
Copper.....	555
Zinc.....	437
Lead.....	710
Tin.....	456
Aluminum.....	166-168
Nickel.....	518-543

For "corrosion cracking" of brass, see Art. 8 of Chap. XIX of the volume on "Strength of Materials," dealing with initial stresses.

For further study of nonferrous alloys consult Mills, Upton, Johnson, Moore, and especially Thurston; and also the recent work, "Metals and Their Alloys," by Charles Vickers: London; Crosby, Lockwood & Son, 1923.

*Circular No. 101 of the U. S. Bureau of Standards* on "Physical Properties of Materials" contains much tabulated information.

## CHAPTER X

### STONE

#### REFERENCES:

MERRILL: "Stones for Building and Decoration."

RIES: "Building Stones and Clay Products."

ECKEL: "Building Stones and Clays."

WATSON: "Engineering Geology."

JOHNSON: Chap. VII.

MILLS: Section 3.

Watertown Arsenal: Reports for 1894-95.

1. Stone has been used as a structural material from time immemorial, generally in compression, never in pure tension, sometimes in flexure, as where a block spans an opening like that for a door or window, or the distance between two columns in ancient temples.

#### 2. Kinds of Stone for Structures.—These are:

*Igneous rocks*: formed by solidification from a fused condition, and crystalline in structure, such as *granite*.

*Sedimentary rocks*: deposited in water, and laminated in structure, such as *sandstone* and *limestone*.

*Metamorphic rocks*: either igneous or sedimentary, which have been modified by heat or pressure, generally crystalline and often laminated, depending on the pressure, such as *gneiss*, *marble* and *slate*. Slate is clay shale which has been consolidated by great pressure.

Stones, even of the same class, differ greatly in strength, hardness, and durability. The strongest and most durable under ordinary conditions are the granites and gneisses, but they are deficient in fire-resisting power.

Sandstone consists of sand cemented by some other material. Some are hard, dense and strong, while others are soft and weak, and will almost crumble in the hand. The same is true of limestones. Before using any stone in important work, unless experience is available regarding it, tests should be made of its crushing strength and durability.

**3. Weight and Strength of Stone** (see "Tests of Materials at Watertown Arsenal," 1894-95).—The following are average values, but there is great variation between different quarries:

STRENGTH OF STONES

Kind	Weight per cubic foot pounds (Eckel)	Ultimate strength, pounds per square inch			Poisson's ratio
		Compression	Shearing	Modulus of rupture in flexure	
Granite.....	168	20,000	2,250	1,600	0.172-0.25
Sandstone.....	157	12,500	1,685	1,450	0.091-0.333
Limestone.....	165	9,000	1,400	1,240	0.27
Marble.....	175	12,600	1,300	1,500	0.222-0.345
Slate.....	174	.....	.....	7,000	

The weights given in the foregoing table are of the stone itself. The weight of masonry will be somewhat less, depending upon the thickness of joints and character of mortar. It may be taken about as follows:

WEIGHT OF MASONRY

Kind of stone	Ashlar masonry	Mortar rubble masonry	Dry rubble masonry
Granite.....	165	155	130
Sandstone.....	150	135	110
Limestone.....	160	150	125
Marble.....	170	160	135

The variability of stone from different quarries must be borne in mind. Thus, granite in compression varies from about 15,000 to 26,000 pounds per square inch; sandstone from less than 7,000 to nearly 20,000; limestone from 3,000 or less to over 20,000; marble, which is more uniform, from about 10,000 to 16,000. For Wisconsin building stone, the modulus of rupture was found to be: for granite, 2,713 to 3,910; for sandstone, from 363 to 1,324; for limestone, from 1,164 to 4,659. Merrill gives, for the compressive strength of sandstone at right angles to the natural bed, 5,481 to 17,500 pounds. The strength parallel to the bed is generally, though not always, less than perpendicular to the bed.

**4. Elasticity of Stone.**—Stone, like brick and concrete, does not deform according to Hooke's law. The strain does not increase proportionally to the stress, but less rapidly, as shown by Fig. 38 for a granite. The curve is at first concave upward, and then nearly straight. This may perhaps be due to the initial compression of the thin layer of plaster which is spread over the ends in testing or, if no plaster is used, to the initial crushing of the particles at the necessarily rough ends until a full bearing is obtained. At all events, the yielding is proportionally greater at first than later, and this initial yielding is mechanical rather than elastic. The shape of the curve is just the reverse of those for wood and cast iron, for which materials the strain increases

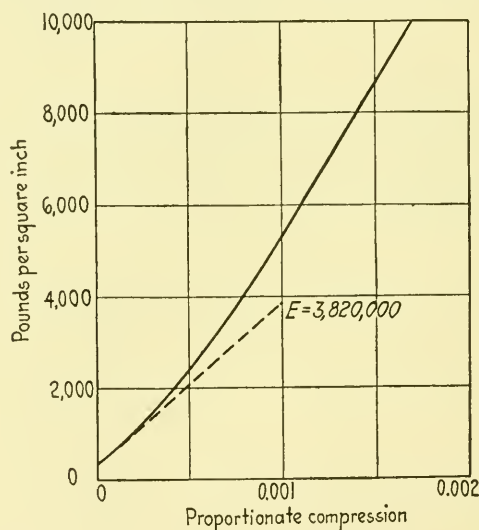


FIG. 38.—Milford pink granite. (*Tests of Metals*, 1894.)

faster than the stress. In stone, the modulus of elasticity increases as the load increases. Values given in different books vary, even for the same material, because they are not always taken at the same point on the curve. If the value is of importance to the engineer, he should consult the records of original tests.

Poisson's ratio for stone seems to average about 0.25, or about the same as for iron and steel, or a little less.

The strength of stone masonry involves the strength of the mortar joints, which are thin, and therefore have greater strength than the figures usually given for mortar. Weakness of the joints could not cause failure, though it might cause settlement.

If the mortar in the joints disintegrates it may cause failure, particularly if the masonry is not laid with coursed joints.

**5. Effect of Shape and Dimensions on Strength.**—Experiments to determine the difference of strength of rectangular prisms, cylinders, and other shapes, and the effect of length, were early made by engineers, and are fully described by Bauschinger.<sup>1</sup> They were generally with small specimens, not over about 1 inch square in section. Vicat stated that for geometrically similar bodies, similarly loaded, the ultimate strength varied as the square of homologous sides, and gave many striking proofs. In his experiments he interposed soft cushions of paper between his specimens and the machine heads, and Bauschinger found the results in some respects entirely different when the end surfaces were smoothed and had no layers of soft material. This illustrates how one detail in testing may make different experiments impossible of proper comparison. Vicat found that as long as the length,  $l$ , was less than that of a cubic specimen the strength was

$$P = \frac{\text{constant}}{l} + \text{constant},$$

and Bauschinger found thin specimens tested flat to be very strong, as would be expected. It was also found that rectangular blocks one over the other without mortar always had considerably smaller strength than single blocks.

Bauschinger gave the formula

$$\frac{P}{A} = \sqrt{\frac{\sqrt{\text{area of section}}}{\text{circumference}}} \left( \text{constant} + \text{constant} \frac{\sqrt{\text{area}}}{l} \right)$$

but reaches the conclusion that the shape of the section is of very minor importance. Hodgkinson reached the same conclusion regarding short lengths of cast iron.

General Gillmore,<sup>2</sup> however, believed that the strength of cubes of stone increased as the cube root of the side. He also studied the effect of cushions of steel, wood, lead, and leather, the results being discordant but generally indicating that the strengths were in the proportion of 100, 89, 65 and 62, respectively. A soft cushion, and especially one that can flow, like lead, and with the stone surface unpolished, no doubt forces its way into the interstices of the stone and diminishes the strength by increasing the tendency to split rather than to shear. Unwin

<sup>1</sup> MITTHEILUNGEN, vol. 6.

<sup>2</sup> Report of the Chief of Engineers, U. S. A., for 1875.



found the strength between lead plates in one case three-fifths and in another three-sevenths of the strength when crushed between unyielding surfaces, the ends of the block being evened up with plaster.

**6. Effect of Loading Part of the Section.**—If, instead of loading the entire end of a stone specimen, the load is applied only to a part of the area, but still centrally, there is an important effect. Bauschinger tested pieces having the shapes shown in Figs. 39*b* and *c*, by which the load was applied to a part of the section at one end, either by bevelling the edges or by a steel block, and to the entire section at the other end, or to equal parts of the section at each end (Fig. 39*d*). Testing without any soft intermediate layer at the ends, he got no splitting of the blocks longitudinally, but in every case the failure was by the formation of a

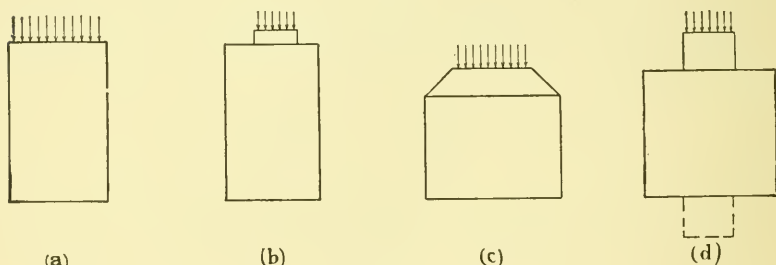


FIG. 39.

pyramid having for its base the small pressure area, and an opposing pyramid not so plainly marked, which wedged off pieces on the sides. The strength is shown in the following statement; with 4-inch cubes of Swiss sandstone bevelled at one end, if the average ratio of small end to large end was 0.182, 0.377, 0.658, the average ratio of the ultimate unit stress on the small end to that for a complete cube was 1.82, 1.33, 1.13; and the ratio of the ultimate unit stress on the large end to that for a complete cube was 0.333, 0.51, 0.726. The angle of bevelling at the end had little influence. Tests of complete cubes or cylinders showed two pyramids, or cones, one at each end; with pieces of greater length the length of the pyramids or cones increased, and with smaller lengths they decreased, and with lengths smaller than lateral dimension the base of the pyramids did not occupy the entire face exposed to compression, as it did for cubes or longer pieces; and the side pieces or plates split off first,

the hour-glass shaped piece remaining intact, and carrying a much greater load.

Tests with load applied over part of one or both ends (Figs. 39*b* and *c*) gave results as follows for cubes of about 4 inches of Swiss sandstone; *with steel block at one end only*, if the ratio of compression area to the full section was 0.154, 0.333, 0.618 the ratio of ultimate on the small compressed area to that for a complete cube was 1.60, 1.40, 1.17 and the ratio of ultimate on the large end to that for a complete cube was 0.247, 0.469, 0.726. *With equal steel blocks at each end*, if the ratio of the compression area to the full section was 0.156, 0.328, 0.630 the ratio of ultimate on the compression area to that for a complete cube was 0.901, 0.749, 0.901 and the ratio of ultimate on full cross-section to that for a complete cube was 0.140, 0.247, 0.574. These last results are very remarkable: with the pressure applied through two equal square blocks of an area smaller than the cross-section, the strength of the cube was only about as great as for a prism having the area of the blocks and the height of the cube, the material outside of this prism doing little good. Vicat had already noticed a similar fact. It is due to the action of pyramids having bases equal to the area of the block, which wedged off side pieces instead of exposing them to compression.

Bauschinger also experimented with eccentric loading applied by a steel block on a narrow area parallel to one side and extending clear across the face. He found the law of planar distribution fulfilled in this case. The failure was by the formation of a narrow wedge under the bearing area (even when that area was close to the side of the cube), which wedged off the sides. He gave a formula for an eccentric load applied over a smaller square or rectangular area anywhere in the face of the block.

In testing stone, it is obvious that the pressure should be applied over the entire end area. It has been explained that the plane on which shearing is greatest is at  $45^\circ$  with the direction of the load, but that taking friction into account the plane on which shearing would be most likely to occur is *steeper than  $45^\circ$* , being as high as  $60^\circ$  for brick or stone. If the angle is  $45^\circ$ , it is obvious that two pyramids cannot form in a square rectangular prism if the piece is shorter than a cube; if  $60^\circ$ , if the height is less than 1.73 times the side. With less height than these limits the strength should increase. Thus, a brick tested flat should give larger strength than if tested on edge or endwise.

In masonry, the function of a mortar joint is to secure even distribution of the load and to keep out water. If a bridge bed plate rests on stone masonry, a thin layer of cement or a plate of lead is generally placed under it.

**7. Allowable Stresses.**—The factor of safety for stone is large, generally from 10 to 20, or even larger for stone of poor quality. It varies with the quality of masonry, being less for the best cut-stone masonry, and larger for poorer masonry. For rubble masonry, in which the stones are of irregular sizes and shapes, and there are no regular joints, the allowable stress may be as low as 150 or 200 pounds per square inch.

The following are figures given by some specifications:

A.R.E.A., for railway bridges, impact being added:

	POUNDS PER SQUARE INCH
Bearing on granite masonry.....	900
Bearing on sandstone and limestone masonry.....	400
Bearing on concrete masonry.....	600

Canadian Department of Railways and Canals, for railway bridges, impact being added, on first class masonry, and on Portland cement concrete, not less than one month old:

	POUNDS PER SQUARE INCH
Sandstone.....	300
Concrete.....	400
Sound limestone.....	400
Granite.....	500

Massachusetts Public Service Commission, for electric railway and highway bridges, impact being added:

	POUNDS PER SQUARE INCH
(Kind of masonry not specified).....	400

These figures may not be properly comparable, because the percentages of impact may not be the same.

Building Code of National Board of Fire Underwriters:

	POUNDS PER SQUARE INCH
Rubble stonework in Portland Cement mortar.....	140
Rubble stonework in lime and cement mortar.....	100
Rubble stonework in lime mortar.....	70
Cut stone masonry, other than sandstone.....	600
Sandstone masonry.....	300
Granites, according to test.....	1,000 to 2,400
Gneiss.....	1,000
Limestones, according to test.....	700 to 2,300
Marbles, according to test.....	600 to 1,200
Sandstones, according to test.....	400 to 1,600
Slate.....	1,000

Allowable stresses for flexure or shearing are not generally specified. If necessary to use them, one-twentieth the ultimate stresses given in Art. 3 may generally be safely used, though Bauschinger found some Bavarian sandstones for which this would be too high. In any important case, the stone should be tested.

Bridge bearings are generally arranged as in Fig. 40.

**8. Durability of Stone.**—The durability of a stone depends upon its composition and structure, and the circumstances of its exposure.

As regards composition, some minerals are very durable, while others weather rapidly. Silicates are generally the most resistant, carbonates much less so. Certain constituents are undesirable. A knowledge of mineralogy is necessary in order to judge of this, and if the matter is important the advice of a geologist or mineralogist should be obtained.

The most powerful disintegrating agencies are water and change of temperature, to which may be added, in certain cases, fire and fumes. If stone is likely to be exposed to particular fumes, the advice of a chemist should be sought as to whether they will have a serious effect. Resistance to fire will be presently referred to. The most important remaining elements, water and temperature, depend mainly on the porosity of the stone and the solubility of its constituents in water. All stones are more or less porous, and will absorb water. The more porous they are, the less desirable. The granites, as a rule, absorb less than 1 per cent of their dry weight, many of them less than one-half of 1 per cent; sandstones frequently ten times as much; limestones and marbles from 1 to 3 or 4 per cent.<sup>1</sup> This absorption is a good index of the durability and ability to resist the action of water and freezing; though something depends upon the cohesive strength to resist a disruptive force, for clearly a force sufficient to disrupt one stone might not disrupt another.

From this point of view, the granites and gneisses are the most durable, the marbles and limestones next, the sandstones last; and experience shows this to be correct as a rule.

<sup>1</sup> See tables in MERRILL.

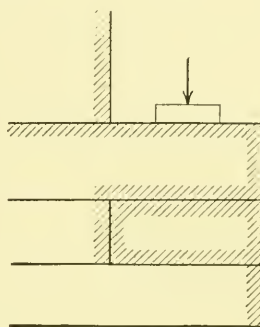


FIG. 40.

In judging between two stones, a test of absorption is often advisable. It may also be desirable to test the resistance to freezing by soaking cubes in water, exposing them to a temperature well below freezing for a time, and repeating the process a number of times; then drying, measuring the loss of weight and observing any signs of disintegration. If freezing temperatures are not obtainable, Brard's test may be used, which consists in immersing in a solution of sodium sulphate, which is allowed to crystallize in the pores. Johnson, however, correctly says that "since the results of this test do not bear any fixed relation to the results of freezing tests, its value is decidedly questionable."

On the whole, experience with a given stone is the best test of its durability.

**9. Resistance to Fire.**—From this point of view the granites are the least resistant, high temperatures causing them to scale and crack badly, owing to the different expansion of the different constituents. From limestones, the carbonic acid is expelled by high heat, and the surface crumbles. Sandstones are generally the most resistant, depending upon the cementing material, being much better than granites, and generally better than limestones. Brick, however, is better than any of these stones.

**10. Protection of Stone.**—Stonework is sometimes coated or painted with a coating of preservative, such as paraffine, linseed oil, soap and alum solutions, or by Ransome's process. The latter consists in first applying a coat of sodium or potassium silicate, and following this, when dry, a coat of calcium chloride, producing a silicate of lime.

**11. Specifications.**—Specifications for stone masonry often prescribe the kind of stone and even the particular quarry. Sometimes the choice of several is allowed. Sometimes it is merely required that the stones shall be satisfactory to the engineer, but this is not sufficiently definite. These methods make it unnecessary to specify the strength. It is generally required that the stones shall be free from seams or other imperfections, and that seasoned stone (that is, stone from which the quarry moisture has been allowed to dry out) shall be used wherever there is liability to frost, since freezing might be quite injurious to unseasoned stone.

The remaining requirements concern the dimensions and arrangement of the stones, the dressing of the surfaces, character of joints, etc., and belong under the subject of Masonry.



## CHAPTER XI

### BRICK AND OTHER CLAY PRODUCTS

#### 1. References:

Chapters in MILLS and in JOHNSON.

SEARLE: "Modern Brick Making."

RIES: "Building Stones and Clay Products."

2. The Clay products used in structures include brick, hollow blocks for building or fireproofing, architectural terra cotta, terra cotta lumber, roofing and other tiles, and pipes for drains, sewers and culverts. These are all made by molding, pressing, drying, and burning clay, the burning often carried to the point of incipient vitrification. The character of the product is very variable, depending upon the raw material, method of manufacture, and particularly the degree of burning.

3. Brick is an important structural material. Well-burned, hard bricks should have a generally uniform color, should be free from cracks, laminations, or blisters, should ring when struck with a hammer, should be so hard that when a brick is broken the interior can with difficulty be scratched with a knife, and should absorb a small percentage of water.

4. The A.S.T.M., in Specification C 21-20, gives the following classification of building brick:

Name of grade	Absorption limits, per cent		Compressive strength (on edge), pounds per square inch		Modulus of rupture, pounds per square inch	
	Mean of 5 tests	Individual maximum	Mean of 5 tests	Individual minimum	Mean of 5 tests	Individual minimum
Vitrified brick....	5 or less	6.0	5,000 or over	4,000	1,200 or over	800
Hard brick.....	5 to 12	15.0	3,500 or over	2,500	600 or over	400
Medium brick....	12 to 20	24.0	2,000 or over	1,500	450 or over	300
Soft brick.....	20 or over	No limit	1,000 or over	800	300 or over	200

The standing of any set of bricks is to be determined by that one of the three requirements in which it is lowest.

The Committee of the A.S.T.M. instituted an elaborate series of tests of bricks. The bricks were tested flatwise in compression, and in flexure flatwise with a span of 6 inches and load at the center. Since a brick measures only about 2 by 4 by 8 inches, there is objection to testing it flatwise in compression, and the results will be above the real strength, since the length of 2 inches is insufficient to allow a complete shear fracture to be developed. More reliable results are obtained by testing them edgewise or endwise. Tests by the U. S. Bureau of Standards<sup>1</sup> showed an average strength when tested flatwise of 6,226, and when tested edgewise of 5,399 pounds per square inch.

Orton states that true vitrification would correspond to an absorption of less than 3 per cent, and that if the absorption is less than 5 per cent, danger from frost is negligible.

Softs bricks are unsuitable for use to carry loads and should only be used for filling. Structural specifications generally call for hard-burned brick. Pressed brick, which have been subjected to heavy pressure in molds after drying and before burning, are more nearly perfect in shape than ordinary brick, but are costly and only used where appearance is important.

**5. Strength of Brick.** *Crushing.* Mills gives average crushing strength about as follows, in pounds per square inch:

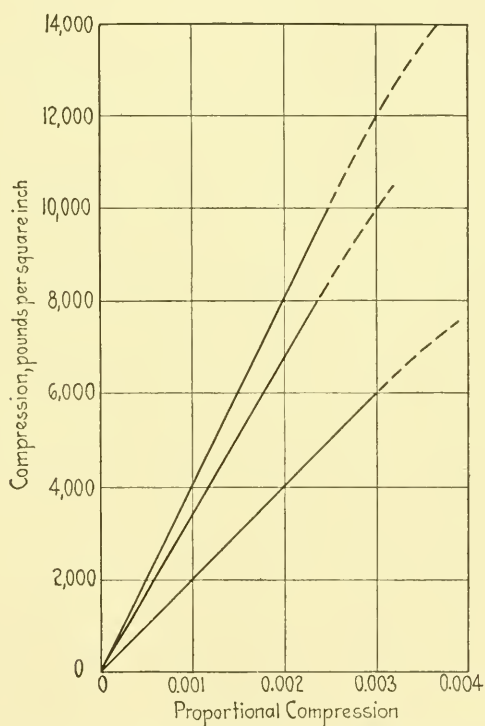
	POUNDS PER SQUARE INCH
Good building brick.....	4,000
Pressed brick.....	8,000
Sand-lime brick <sup>1</sup> .....	3,000 to 4,000
Paving brick.....	10,000
Fire-clay brick.....	3,000 to 6,000
Terra cotta blocks.....	4,000
Architectural terra cotta.....	3,000

<sup>1</sup> Sand-lime bricks are not made from clay, but from a mixture of slaked lime and sand, molded and hardened under steam pressure.

*Transverse and Shearing Strength.*—The tests at Watertown Arsenal indicate, according to Mills, about the following values for modulus of rupture and shearing strength:

<sup>1</sup> *Proc. A.S.T.M.*, p. 150, 1915. See also paper by EDWARD ORTON, JR. in vol. XIX, p. 268, 1919.

Kind	Modulus of rupture, pounds per square inch	Shearing strength, pounds per square inch
Common building brick.....	500 to 1,000	1,000 to 1,500
Pressed brick.....	600 to 1,200	800 to 1,200
Sand-lime brick.....	300 to 600	500 to 1,000
Paving brick.....	1,500 to 2,500	1,200 to 1,800
Fire-clay brick.....	300 to 600	500 to 1,000
Unglazed terra cotta blocks.....	500 to 1,000	

FIG. 41.—Common brick. (*Tests of Metals*, 1885.)

**6. Elasticity of Brick.**—Brick does not follow Hooke's law, and yet tests at Watertown Arsenal indicated a close adherence to that law up to loads of 6,000 to 10,000 pounds per square inch.

Three grades of brick were tested, on end. The strongest averaged 14,000 pounds per square inch and showed an almost constant value of  $E$  of 4,100,000 pounds per square inch up to 10,000 pounds per square inch; the next grade averaged 10,500, and showed, up to 6,000,  $E$  nearly constant and equal to 3,400,000; the third grade averaged 7,500, and showed  $E$  equal to 2,000,000 up to 6,000.<sup>1</sup> Softer grades would no doubt show smaller values of  $E$ , and diagrams convex upward, like those for concrete. Figure 41 shows the stress-strain diagram.

**7. Weight.**—The specific gravity of brick is given by Johnson as from 1.9 to 2.6, according to the character of the raw materials and the degree of burning. This corresponds to a weight per cubic foot of 118 to 162 pounds.

The weight of *brickwork* will depend also upon the character of mortar and thickness of joints, as in other masonry, and may be taken as from 100 to 140 pounds per cubic foot.

**8. Strength of Brick Piers.**—This will depend not so much on the strength of the bricks alone, as upon the bond (arrangement of bricks in the courses), the character of the mortar and thickness of joints, and the workmanship. A pier generally will fail by breaking up into smaller columns by transverse fracture of individual bricks. This will depend also upon the size of the column and the character of bond possible, and the regularity in form of the brick, which will affect the tendency to break when laid up in a pier. A building brick measures approximately 8 by 4 by 2 inches, and is laid flat. If the side (8 by 2 inches) shows on the face, the brick is called a *stretcher*; if the end (4 by 2 inches) shows, it is a *header*. If all faces showed stretchers only (except the end bricks) the pier would consist of an outer shell 4 inches thick, not tied to the inside, and a load on this shell would not be distributed. There should be a certain proportion of headers. In common bond, all are stretchers for a certain number of courses, and then there is a course entirely of headers. In English bond courses of stretchers and headers alternate. In Flemish bond each course has alternate headers and stretchers.

The strength of brick piers depends upon the character and strength of the brick, the method of bonding, the character and strength of the mortar, the age, the dimensions, and the workmanship. The number of variables explains the divergent results. *Technologic Paper* 111 of the Bureau of Standards gives

<sup>1</sup> "Tests of Metals," p. 1138, 1885.

## TESTS OF BRICK PIERS, T.P. 111 U. S. BUREAU OF STANDARDS

Number	District	Grade	Bond	Piers				Brick				
				Mortar	Height, ft.	Area, sq. in.	Age, days	Max. unit load lbs. per sq. in.	$f_c$ lbs. per sq. in.	$f'_c$ lbs. per sq. in.	$f_b$ lbs. per sq. in.	Per cent absorption
1	P	1	1:1	A	10	930	30	2,710	11,990	8,900	1,945	4.08
2	P	1	1:3	A	10	930	30	2,740	11,990	8,900	1,945	4.08
3	P	1	1:6	A	10	930	30	2,900	11,990	8,900	1,945	4.08
4	P	2	1:1	A	10	856	32	2,000	7,880	6,450	1,370	7.46
5	P	2	1:3	A	10	885	32	2,070	7,880	6,450	1,370	7.46
6	P	2	1:6	A	10	946	33	870	2,450	2,040	675	15.16
7	P	3	1:1	A	10	1,024	29	510	1,659	1,350	345	16.28
8	P	3	1:3	A	10	1,043	29	560	1,659	1,350	345	16.28
9	P	3	1:6	A	10	1,024	32	650	1,659	1,350	345	16.28
10	P	1	1:1	C	10	841	90	3,800	11,965	10,050	2,775	1.28
11	P	1	1:3	C	10	841	90	3,220	11,965	10,050	2,775	1.28
12	P	1	1:6	C	10	841	90	3,300	11,965	10,050	2,775	1.28
13	P	2	1:1	C	10	908	33	1,760	7,880	6,450	1,370	7.46
14	P	2	1:3	C	10	961	32	870	2,450	2,040	675	15.16
15	P	2	1:6	C	10	878	30	1,760	7,880	6,450	1,370	7.46
16	P	1	1:1	B	10	940	120	1,450	11,990	8,900	1,945	4.08
17	P	1	1:3	B	10	940	120	1,270	11,990	8,900	1,945	4.08
18	P	1	1:6	B	10	940	120	1,360	11,990	8,900	1,945	4.08
19	P	2	1:1	B	10	906	120	840	7,880	6,450	1,370	7.46
20	P	2	1:3	B	10	906	120	890	7,880	6,450	1,370	7.46
21	P	2	1:6	B	10	900	120	990	7,880	6,450	1,370	7.46
22	P	3	1:1	1 lime; 3 sand	10	1,024	120	210	1,659	1,350	345	16.28
23	P	3	1:3	1 lime; 3 sand	10	1,024	120	178	1,659	1,350	345	16.28
24	P	3	1:6	1 lime; 3 sand	10	1,024	120	126	1,659	1,350	345	16.28
25	O	1	1:1	C	10	841	29	1,450	7,340	4,910	733	16.80
26	O	1	1:3	C	10	841	29	1,760	7,340	4,910	733	16.80
27	O	2	1:1	C	10	841	31	1,630	6,880	5,490	893	16.40
28	O	2	1:3	C	10	841	30	1,790	6,880	5,490	893	16.40
29	O	3	1:1	C	10	841	30	1,880	6,510	5,700	1,090	17.10
30	O	3	1:3	C	10	841	29	1,690	6,510	5,700	1,090	17.10
31	O	3	1:6	C	10	841	31	1,660	6,510	5,700	1,090	17.10
32	Y	1	1:1	C	10	791	32	1,170	5,630	6,440	601	16.40
33	Y	1	1:3	C	10	791	32	1,300	5,630	6,440	601	16.40
34	Y	1	1:6	C	10	791	31	1,260	5,630	6,440	601	16.40
35	Y	2	1:1	C	10	784	30	1,280	4,430	5,449	616	18.60
36	Y	2	1:3	C	10	791	31	1,280	4,430	5,449	616	18.60
37	Y	2	1:6	C	10	791	31	1,220	4,430	5,449	616	18.60
38	Y	3	1:1	C	10	791	31	1,070	2,710	2,970	497	19.30
39	Y	3	1:3	C	10	791	31	1,060	2,710	2,970	497	19.30
40	Y	3	1:6	C	10	791	30	1,020	2,710	2,970	497	19.30
41	C	1	1:1	C	10	841	32	840	3,200	3,010	1,180	16.20
42	C	1	1:3	C	10	812	29	790	3,200	3,010	1,180	16.20
43	C	1	1:6	C	10	812	30	810	3,200	3,010	1,180	16.20
44	C	2	1:1	C	10	812	30	750	3,150	2,710	1,140	16.20
45	C	2	1:3	C	10	812	30	700	3,150	2,710	1,140	16.20
46	C	2	1:6	C	10	812	30	710	3,150	2,710	1,140	16.20
47	P	2	1:1	C	5	915	30	1,730	2,450	2,040	675	15.16
48	P	2	1:1	C	5	915	30	1,840	2,450	2,040	675	15.16
49	P	2	1:1 <sup>1</sup>	C	5	924	30	2,270	2,450	2,040	675	15.16
50	P	2	1:1 <sup>2</sup>	C	5	930	30	1,470	2,450	2,040	675	15.16

<sup>1</sup> Galvanized iron wire mesh in every joint.<sup>2</sup> Galvanized iron wire mesh in every fourth joint.



a review of previous work, and the results of 50 tests of piers about 30 by 30 inches by 10 feet, with a thin cushion of plaster of Paris to even up the ends, with different mortars and bond, and three grades of brick from different parts of the country.

Grade 1 was "hard-burned or best quality."

Grade 2 was "medium-burned or considered as common."

Grade 3 was "soft-burned or poorest product marketed."

The mortars used were:

*A*: 1 part Portland cement to 3 parts sand.

*B*: 1 part hydrated lime to 6 parts sand.

*C*: 1 part composed of 15 per cent hydrated lime (dry weight) and 85 per cent Portland cement (dry weight) to 3 parts sand by weight.

The table on page 147 gives the results of the tests; *P* meaning brick from the Pittsburgh district, *O* the New Orleans district, *Y* the New York district and *C* the Chicago district. The three columns before the last give  $f_c$  = average compressive strength of the bricks flat,  $f_c'$  on edge,  $f_b$  the average modulus of rupture. The age is in days. The ratios of bond are header to stretcher courses; thus 1:3 is one header course to 3 stretcher courses. A study of these tests is illuminating; thus the low strength of No. 14 is explained by the low strength of the bricks and the large percentage of absorption; clearly this figure should not be averaged with those just above and below as the strength of Grade 2 piers (though this is done in some abstracts of these tests).

It was found from these tests that there was an approximately linear relation between the strength of the piers and the transverse and the compressive strength (flat or on edge) of the brick, expressed by the relation

$$\text{max. unit strength of pier (per square inch)} = kf$$

where  $f$  is either  $f_c$ ,  $f_c'$ , or  $f_b$ . The values of  $k$  are given in the following table:

RELATION BETWEEN STRENGTH OF BRICK PIER AND TRANSVERSE AND COMPRESSIVE STRENGTH OF BRICK

Kind of stress	How tested	Mortar	$k$
Transverse.....	Flat, 7-inch span	<i>A</i>	1.45
Transverse.....	Flat, 7-inch span	<i>C</i>	1.25
Transverse.....	Flat, 7-inch span	<i>B</i>	0.65
Compression.....	Flat	<i>A</i>	0.27
Compression.....	Flat	<i>C</i>	0.26
Compression.....	Flat	<i>B</i>	0.11
Compression.....	On edge	<i>A</i>	0.32
Compression.....	On edge	<i>C</i>	0.30
Compression.....	On edge	<i>B</i>	0.14

The modulus of elasticity varied approximately as the strength of the pier, for cement and cement-lime mortars. Its maximum value was 3,500,000 for tests 10 and 12, and its minimum 270,000 for test 14; the lowest for mortar *A* was 533,000 for test 9, and for mortar *C* 484,000 for test 38.

The primary failure was found to arise from transverse failure of individual bricks; this shows the importance of thin joints of uniform thickness, and uniform bedding of the bricks. Porous bricks must be well wet down. The tests show that 15 per cent of cement (by weight, or about 35 per cent by volume) may be replaced by hydrated lime without lessening the strength, and in some cases increasing it. This is probably due to the fact that the mortar of cement-lime is easier working than mortar of pure cement, and that better bedding is the result. There is a slight increase of strength if wire mesh is used, preferably in every joint.

In *Bull. No. 27* of the University of Illinois, tests are described of 16 brick piers and 16 piers of terra cotta blocks. Of the brick piers, 14 were of shale building brick, a high-grade, hard-burned material with under 3 per cent absorption; and 2 of under-burned clay brick with about 17 per cent absorption, softer than would be used in building construction; the two thus representing extremes in quality. The following table gives a summary of the results:

## BRICK PIERS

Bulletin No. 27, University of Illinois, 1908

Brick	Characteristics of columns, 12½ by 12½ inches by 10 feet approximately	Age	Average ultimate strength; usually average of 2 tests pounds per square inch	Ratio of strength of column to strength of brick flatwise
Hard-burned shale	Well-laid; 1 P.C.; 3 sand	67 days	3,365	0.31
	Well-laid; 1 P.C.; 3 sand	6 months	3,950 <sup>1</sup>	0.37
	Well-laid; 1 P.C.; 3 sand load 1 inch eccentric	68 days	2,800 <sup>2</sup>	0.26
	Poorly-laid; 1 P.C.; 3 sand	69 days	2,920	0.27
	Well-laid; 1 P.C.; 5 sand	65 days	2,225	0.21
	Well-laid; 1 Nat. cem; 3 sand	66 days	1,750	0.52
	Well-laid 1 lime; 2 sand	66 days	1,450	0.14
Underburned clay	Well-laid 1 P.C.; 3 sand	63 days	1,060	0.31

<sup>1</sup> In one of these columns the load was repeated seven times before failure occurred.<sup>2</sup> Computed stress at inside is 4,150 pounds per square inch

Engineering Paper No. 12 of Columbia University, Apr., 1923, by A. H. Beyer and W. J. Krefeld, contains an account of an extensive series of tests of brick piers, 135 piers in all, all 12 inches square, 115 being 40 inches high and 20 being 84 inches high. All variables were kept constant except the kind of brick, the mortar being 1:3 Portland cement. Clay brick was used in 44 piers, concrete brick in 79, sand-lime brick in 8, and monolithic concrete in 4, all in common use in New York City. As in the Bureau of Standards tests, the ratio  $k$  of ultimate compressive strength of pier to ultimate compressive strength of clay bricks appeared to be nearly constant, but different in different series of tests in which the modulus of rupture of the brick and the strength of the mortar differed. For concrete brick the ratio  $k$  decreased slightly as the compressive strength of the brick increased. The concrete brick piers generally failed by diagonal shear. The reader should consult this interesting paper.

The reader should also consult the Watertown tests. They are described in the above-mentioned *Bull. No. 27*.

**9. Allowable Load on Brickwork.**—No generally applicable figure for this can be given, as so much depends upon conditions. The following are given in the Building Code recommended by the National Board of Fire Underwriters for hard-burned brick with an average compressive strength of 3,000 pounds per square inch tested flatwise:

POUNDS PER  
SQUARE INCH

Brickwork in portland cement mortar.....	250
Brickwork in natural cement mortar.....	208
Brickwork in lime and portland cement mortar.....	208
Brickwork in lime mortar.....	111

Cement and cement-lime mortar is to be not weaker than 1:3; lime mortar not weaker than 1:4.

*Terra Cotta*.—*Terra Cotta* is made like brick and in a great variety of forms. Hollow blocks are used for floor arches, partitions, walls, columns, fireproofing, etc. *Terra cotta* lumber is

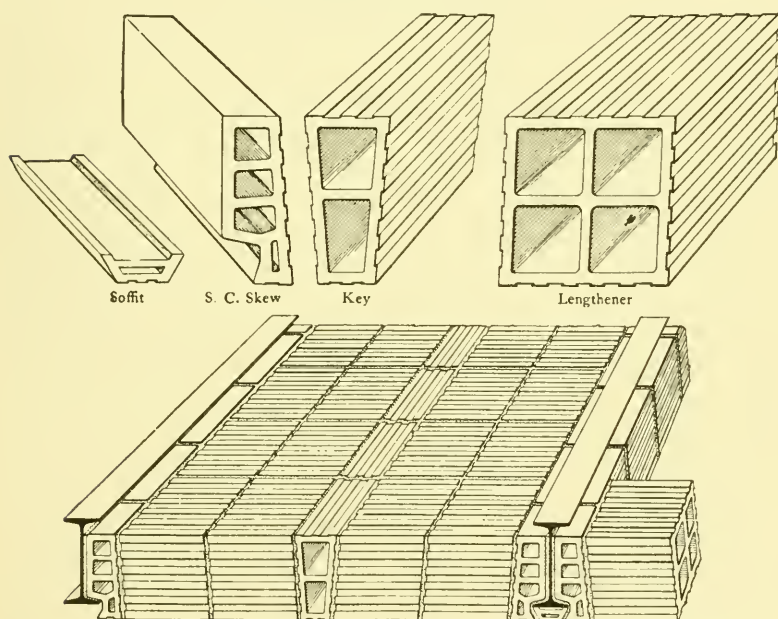


FIG. 42.

(National Fire-Proofing Co.)

made from clays mixed with finely cut straw or sawdust. The straw or sawdust is consumed in burning, leaving a light and porous material into which nails and screws may easily be driven, and which can be cut with a saw. Hollow blocks and fireproofing are made from clay without sawdust, and resemble brick. The strength of these products is very variable. Tests of hollow blocks, loaded on end, described in *Bull. No. 27, University of Illinois* gave 3,350 to 9,070 pounds per square inch of *net* section,

averaging 5,450. These blocks measured about 4 by  $8\frac{1}{2}$  by 8 inches. The strength of columns 10 or 12 feet high varied from 2,710 to above 3,800 pounds per square inch of *gross section*.

Figures 42 and 43 give some forms of terra cotta blocks. The reader should consult Sweets' Engineering Catalogue and

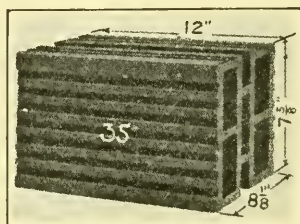


FIG. 43.

Sweets' Architectural Catalogue, very valuable works, for advertisements and cuts by the various companies which manufacture clay products. These products are becoming more and more useful each year.



## CHAPTER XII

### CALCAREOUS CEMENT AND CONCRETE<sup>1</sup>

1. A cement is a material which in a plastic, amorphous, or liquid form, fills the voids between a mass of particles, or is placed between and coating two surfaces, and which, by afterwards hardening, binds those particles or surfaces together. It does this either by virtue of its adhesion to the surfaces—as where it is used to cement together two pieces of glass or crockery or wood—or also by virtue of the fact that it coats and surrounds and fills the voids between a mass of particles like a mass of sand or gravel, so that the mass cannot be separated, or any particle dislodged (except a particle on the very surface) without breaking the hardened cement. We have seen how the theory of amorphous material in steel assumes that the voids between the crystals are filled with this material, which acts like a cement, though it is of the same character as the material of the crystals themselves. Generally, however, cements are very different in character from the particles they serve to bind together.

A cement may harden in several ways. It may harden with water by forming crystals out of its own constituents; in this case there will be voids between these crystals, which should, for the best results, be filled with some amorphous (*i.e.*, uncrystalline) material, like the amorphous material in steel. It may harden by absorbing constituents from the air or from the materials it surrounds, and so form crystals or amorphous material. Or, it may harden without crystallizing, as glue or glass harden, remaining amorphous or colloidal. (A colloid is a non-crystalline or amorphous substance, often or generally glue-like or gelatinous,

<sup>1</sup> For an excellent and clear introduction to the subject of Cement, the reader is advised to read Chap. XX of UPRON'S "Materials of Construction." The writer knows of no better introduction to the study of the more extended treatises than this. In addition to the other treatises that have been referred to, the reader should consult ECKEL, "Cements, Limes, and Plasters," 2d edition, 1922, for an excellent treatment of this whole subject.

One of the very best works dealing with this subject is "A Treatise on Concrete, Plain and Reinforced" by F. W. TAYLOR and S. E. THOMPSON, published by John Wiley & Sons, Inc.

which, however, may harden and become quite strong, its particles being very finely divided. Some writers on cements mention amorphous material, while others mention colloidal material, meaning the same thing.)

There are many kinds of cement, used for uniting many kinds of material. Cements used in engineering are almost universally calcareous, that is, they have lime as a main constituent; and this chapter deals only with these. They are produced from some form of limestone, including chalk and marl.

2. Limestone, if pure, is carbonate of lime ( $\text{CaCO}_3$ ). No limestones are pure, but all contain foreign ingredients, sometimes iron compounds, and often magnesium. Magnesia, belonging to the same chemical class as lime, acts like it in many ways, as in cements, but is inferior and in some respects injurious, as will be seen.

By heating limestone, the  $\text{CO}_2$  is driven off and  $\text{CaO}$ , or *quicklime*, remains. It comes from the kiln in lumps, which are crushed or ground. Quicklime has the property of *slaking*, or uniting with water to form hydrated lime ( $\text{CaO}$ ,  $\text{H}_2\text{O}$ ), which it does with great evolution of heat, and expansion to more than double its former volume. Magnesian quicklimes (as they are called) slake much more slowly than calcium quicklime—so slowly that if the quicklime contains much magnesium it may not slake properly before using. This is one of the sources of danger when cement contains much magnesia, as will be seen. Quicklimes are purposely slaked before being used for mortar, sometimes where the construction work is being done, sometimes at the factory before being shipped, the latter product being known specifically as “hydrated lime.” It is a dry powder, and is the same as quicklime slaked on the work, except that when slaked on the work there is danger, through carelessness or incompetent supervision, of burning or of incomplete hydration, which dangers should not exist when hydrated lime is prepared as a manufactured product. “Hydrated lime” should therefore be superior to ordinary slaked quicklime, and it is much used.

3. Quicklime, on slaking with water, forms either crystalline or amorphous calcium hydroxide, depending mainly upon the fineness and the degree of burning. Crystalline material forms when the hydration is slow, amorphous material when it is rapid. The finer the lime, the more rapid the hydration, and the greater the proportion of amorphous material. The lower the burning temperature,

the more rapid the hydration, and the greater the proportion of amorphous material. The less water used in mixing, the less crystalline material, since the crystals form from solution. Quicklime merely exposed to the air will slake, by absorbing moisture, to the amorphous condition, the expansion in slaking causing the lumps to become powder.

Lime hydrate, in the presence of moisture, combines with  $\text{CO}_2$  from the air, liberating water which evaporates, and becomes limestone, from which it came originally, thus completing the cycle. The hardening of lime mortar, therefore, consists first in the hydration to a more or less hard hydrate, and afterward the slow conversion of this hydrate to limestone. The interior of a masonry joint of lime mortar may never become limestone, for the penetration of the  $\text{CO}_2$  is slow. In the conversion to carbonate, a large contraction occurs, so that large cracks would be formed. To prevent this and also for economy, lime mortar is made by adding a considerable proportion of sand to the paste of hydrated lime.

4. Mortar from lime slaked on the work is made by adding sand to the wet plastic hydrate. Mortar made from hydrated lime is made by mixing the sand with the dry hydrate powder, and then adding water; in this way a more homogeneous mixture is obtained. The mortar from hydrated lime may be made immediately; while quicklime slaked on the work must be allowed to season for at least a day and often much longer, to secure complete hydration. Hydrated lime is more crystalline than ordinary slaked lime, and hence will not bear so large a proportion of sand as the latter; it gives a less plastic mortar, which does not work as smoothly as that from ordinary slaked lime. Being more crystalline, however, the mortars from "hydrated lime" are stronger, shrink less, and set more quickly.

Lime mortars are much used for ordinary buildings, where much strength is not necessary.

Lime mortar will not set under water, for the hydrate is soluble in water, and would be washed out. It is, therefore, not a *hydraulic* material.

**5. Strength of Lime Mortars.**—The strength of lime mortar is very variable. The greater the proportion of sand, the less the strength, as a rule. Fine sand makes a stronger mortar than coarse sand, and magnesium limes make a stronger mortar than calcium limes. The strength increases with age. It decreases

with the amount of water used in mixing. Mills (pages 1, 21) gives a table of tests which may be referred to. The compressive strength may vary from 100 to 400 or 500 pounds per square inch. The Committee of the A.S.T.M. proposes a specification requiring the tensile strength of a mortar of one part by weight of lime and three parts by weight of standard Ottawa sand (see Art. 15) to be not less than 15 pounds per square inch after 7 days in air.

The compressive strength of lime mortar, tested in cubes, even with one part of lime to four of sand, should not be less than 100 pounds per square inch, and may be much greater. A thin joint of mortar would have much greater strength than a cube, and if the mortar has a strength of 100 pounds per square inch tested as a cube, a joint 9 by  $\frac{1}{2}$  inches, in which the ratio of height to width is 18, may have a strength of 1,800 pounds per square inch, and so be able to support the weight of a quite high wall.

**6. Cement.**—Lime is not “cement,” though it is “a cement.” Calcareous cementing materials are classified as common lime, hydraulic lime, natural cement, Portland cement, and slag or puzzolana cement. A hydraulic material is one which will set under water.

Common limes have just been described. They are not hydraulic.

It was long ago found that by mixing common lime with a certain amount of argillaceous material, a product would result which, with proper manufacture, would have hydraulic properties. This is due to the fact that, if intimately mixed and burned, certain calcium silicates and calcium aluminates are formed, which when mixed with water, will set and harden in air or under water. There are no such compounds in ordinary lime mortar, because there is no silica or alumina. These silicates and aluminates hydrate with water by taking up water and forming crystalline hydrates, with some amorphous hydrates, which set quickly compared with lime, and are insoluble in water, and so are able to set under water. To the extent to which the mixture contains more lime than combines with the silica and alumina (and iron if that element is present)<sup>1</sup> there is *free lime*, which hydrates as already explained. The differences in hydraulic limes and cements arise, therefore, from the differences in character and

<sup>1</sup> NEWBERRY states that “iron oxide combines with lime in the same manner as alumina.”

amount of the argillaceous components, and the degree of burning.

*Hydraulic limes* are made by burning slightly argillaceous limestones at a low temperature. They will slake slowly, and have feeble hydraulic properties. They are little used, as they are weak, and set slowly.

*Natural cements* are made by burning distinctly argillaceous limestones at a comparatively high temperature, sufficient to cause the complete combination of lime with silica and alumina into silicates and aluminates. They may have free lime, and if so this will slake, but the cement as a whole cannot be said to slake. They will set under water.

*Puzzolana or Slag Cements* are made by mixing slaked lime with granulated blast-furnace slag, or a natural volcanic ash called puzzolana, without subsequent burning. The slag or puzzolana are silicious or argillaceous. The product will not slake, and has hydraulic properties. It must not be confused with the product obtained by mixing hydrated lime with blast-furnace slag, and then burning and grinding, which is a true Portland cement.

German "Iron Portland Cement" is a mixture of 70 per cent Portland Cement with 30 per cent of granulated blast-furnace slag.

*Portland cement* is what is now generally understood when hydraulic cement is referred to, for its manufacture has increased greatly and its strength and other good qualities render it the most satisfactory calcareous cementing material. The rest of this chapter refers to Portland cement. It is defined as the product obtained by finely pulverizing the clinker produced by burning to incipient fusion an intimate and properly proportioned artificial mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum. Clinker is the material as it comes, in lumps, from the furnace. The main difference between natural and Portland cement is that the former is obtained by burning a natural rock and the latter by burning an artificial mixture. If the mixtures were the same, and of the same homogeneity, in the two cases, the two would be the same, if burned at the same temperature and ground to the same fineness. The calcareous materials used may be pure limestone, chalk, or marl, or hydrated lime. The argillaceous materials may be clay, slate, or shale, impure limestones containing clay, or blast-furnace slag. Know-



ing the chemical composition of the raw materials, they may be mixed in proper proportion to produce the silicates or aluminates required in the cement, leaving little or no free lime or free clay. The raw materials are crushed, ground, mixed in proper proportion, and then burned in a revolving kiln. The clinker comes out in lumps, and is then ground into cement. The silicates and aluminates would set too fast for practical purposes, and so plaster of Paris or gypsum is added to retard the set. Gypsum is a sulphate, which is injurious if in too large quantity, so that the amount of retarder added is limited.

7. A typical average analysis of Portland cement would be approximately, taking a well-known brand as typical:

	PER CENT
SiO <sub>2</sub> .....	21.08
Al <sub>2</sub> O <sub>3</sub> .....	7.86
Fe <sub>2</sub> O <sub>3</sub> .....	2.48
CaO.....	63.68
MgO.....	2.62
SO <sub>3</sub> .....	1.25

When mixed with a suitable amount of water, such a cement becomes hard, or *sets*, in a few minutes, and afterward continues to *harden* for a long time.<sup>1</sup> This is the result of the hydration and crystallization of the silicates and aluminates, and the formation of some amorphous material which gradually hardens as glue or melted glass hardens, and which may slowly crystallize.

8. The important physical properties of cement, for the engineer, are specific gravity, fineness, time of setting, soundness, and strength.

9. **Specific Gravity.**—This quality was formerly considered useful as a means of detecting adulteration and underburning, since the specific gravity of underburned cements is slightly less than that of hard-burned cements. The difference, however, is very small, and not of importance. The test may detect adulter-

<sup>1</sup> The setting and hardening of hydraulic cements has been carefully studied by chemists. The reader may pursue the subject in the following readily accessible sources:

1. TAYLOR and THOMPSON: Chap. VI, by S. B. Newberry.

2. KLEIN and PHILLIPS: "Hydration of Portland Cement"; *Technologic Paper* 43, U. S. Bureau of Standards, 1914.

3. BATES and KLEIN: "Properties of Calcium Silicates and Calcium Aluminate Occurring in Normal Portland Cement," *Technologic Paper* 78, U. S. Bureau of Standards, 1917.

ation, if considerable and with a substance whose specific gravity is different from that of the cement; but the other tests would in such case probably show unsuitable material. A reduction of specific gravity may be due to seasoning, or hydration of free lime and absorption of  $\text{CO}_2$  by exposure to the air; but hydration of free lime previous to use is desirable. The importance of specific gravity is therefore small. Standard specifications provide that the specific gravity shall not be less than 3.10, but that the test will not be made unless specifically ordered. The test is made by dropping a given weight of cement into a glass flask filled with kerosene or benzine up to a given mark on its long graduated stem, and noting the rise of the liquid in the stem. This gives the volume of liquid displaced, or the volume of a given weight of cement, from which the specific gravity is found.

**10. Fineness.**—The standard A.S.T.M. specification requires that the residue on a No. 200 sieve shall not exceed 22 per cent by weight. A No. 200 sieve is one having 200 wires per inch (or between 192 and 208), the diameter of wire being 0.0021 inch (or between 0.0019 and 0.0023), so that the average opening between wires is 0.0029 inch. The openings will not be uniform, but must not exceed 0.005 inch.

It was long ago shown that the coarse particles of cement are inert, and scarcely more effective than so much sand; notably by Eliot C. Clarke in his experiments made while building the main drainage works of Boston,<sup>1</sup> and by many experiments since. The impalpable flour is the most effective cementing material, but the exact size at which a particle becomes inert is not known. This is because the coarser particles are not easily accessible to water throughout, and so do not easily crystallize into the hydrated silicates and aluminates. The outside of coarse particles may be partly vitrified, but even if not, the chemical action on the outside seals, as it were, the inside, and prevents or retards action there. Obviously, then, the strength of a mortar of a given cement will increase with the fineness, though different cements of the same fineness will not show the same strength. Cements of average fineness will be well within the prescribed limits, and will leave a residue of only 10 or 15 per cent, while some leave but 6 per cent, and very fine cements may leave but

<sup>1</sup> See *Trans. A.S.C.E.*, vol. XIV, p. 141, 1885. See also DUFF A. ABRAMS, *A.S.T.M.*, 1918.

2 per cent. Fine grinding, however, is expensive, and it is a question what standard of fineness is most economical.

Fine grinding promotes soundness (see Art. 12) by favoring complete and rapid hydration. For the same reason, it makes a cement more quick setting. Since the coarse particles are inert, it also increases the sand-carrying capacity of a cement; in other words, the greater the fineness, the greater the proportion of sand that may be mixed with the cement to produce a mortar of given strength, or the greater the strength of a mortar with a given proportion of sand.<sup>1</sup> Fine grinding is more effective in increasing the strength at 7 days than at 28 days or longer periods. It lowers the weight per cubic foot about  $\frac{3}{4}$  pound for each 1 per cent that the residue on the No. 200 sieve is reduced; because if coarse particles are ground fine, the whole is made up more nearly of particles of the same size, and smaller ones do not fill the voids between larger ones, so that the percentage of voids is increased.

Fine grinding is obviously desirable to as great an extent as is compatible with economy.

**11. Time of Setting.**—The setting time has no general significance. For some purposes a quick-setting cement is desirable, and for other purposes a slow-setting cement. Standard specifications prescribe that the initial set shall not develop in less than 45 minutes when determined by the Vicat needle, or 60 minutes when determined by the Gillmore needle;<sup>2</sup> and that final set shall be attained within 10 hours. After final set, cement continues to *harden* for a long time. There is no relation between setting time and strength at 7 or 28 days.

The setting time, as tested, depends upon the temperature, the manipulation in testing, and particularly upon the plasticity of the pat tested, so that it is necessary to make the test always with a standard amount of water used in mixing the cement, or

<sup>1</sup> Fine grinding often lowers the neat strength, while increasing the strength of mortars.

See MEADE, in *A.S.T.M.*, p. 408, 1908.

See also ABRAMS: "Effect of Fineness of Cement," *A.S.T.M.*, pt. II, p. 328, 1919.

<sup>2</sup> The U. S. Gov't specification specifies only the Gillmore needles, and puts the initial set at not less than 45 minutes.

The Gillmore needle is named after Gen. Q. A. Gillmore, U. S. A., one of the early students of Cement, who in 1863 published a book entitled "Practical Treatise on Limes, Hydraulic Cements and Mortars," probably the earliest extended treatise on the subject in the United States.

*gaging*, as this determines the plasticity. A wet mixture will obviously set slower than a dry mixture ("mix," as it is called). Seasoning sometimes increases and sometimes decreases the time of set.

The test of setting time is made by mixing a batch of "neat" cement paste (*i.e.*, with no admixture of sand) with a standard amount of cement and of water, and (when using Gillmore needles) forming a pat about 3 inches in diameter and  $\frac{1}{2}$  inch in thickness, and keeping it in moist air at a temperature as near 70° F. as practicable. The Gillmore needles are, one with a

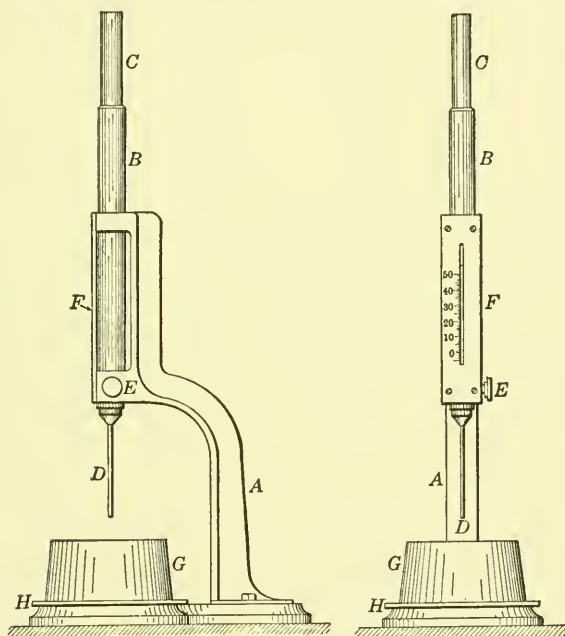


FIG. 44.

diameter of  $\frac{1}{12}$  inch loaded with a lead weight of  $\frac{1}{4}$  pound, and the other with a diameter of  $\frac{1}{24}$  inch loaded to weigh 1 pound. The initial set is assumed to be reached when the pat will bear, "without appreciable indentation," the larger needle; and the final set when it will bear the smaller and heavier needle without appreciable indentation. It is not easy to tell what is meant by "appreciable indentation," and this has led to the use of the Vicat apparatus, in which a needle 1 millimeter in diameter and 6 centimeters long is attached to the end of a larger rod weighing

300 grams and mounted in a frame with a graduated scale (see Fig. 44). The pat is molded in a hard-rubber ring 7 centimeters in diameter at the base and 4 centimeters high resting on a glass plate, and placed under the needle, which is carefully brought in contact with the surface. Initial set is said to have occurred when the needle does not penetrate below a point 5 millimeters above the glass plate in  $\frac{1}{2}$  minute after being released; and final set, when the needle "does not sink visibly into the paste."

Normal consistency of paste is also determined with the Vicat apparatus, and is defined as that consistency when the Vicat rod 1 centimeter in diameter (not the needle) sinks to a point 10 millimeters below the original surface in  $\frac{1}{2}$  minute after being released.

**12. Soundness.**—A cement is unsound if it expands after setting, thus causing cracking or disintegration. Soundness is obviously a most important and necessary quality. The principal cause of unsoundness is free lime which does not hydrate until after set has occurred.

The presence of free lime may be due to an excess in the original composition, above the amount necessary to form the silicates and aluminates which cause the set; or to failure to mix the raw materials before burning so as to form a homogeneous mixture; or to a failure to burn at a temperature sufficient to form the silicates and aluminates.

If the free lime were in its ordinary condition and accessible to the water used in mixing, it would hydrate at once, before setting. Failure to so hydrate may be due to coarse grinding, the water of mixing gradually penetrating the coarse particles and hydrating the free lime there after setting has occurred, especially if the surface of the coarse particles is partly coated with vitrified clinker. Soundness is thus promoted by fine grinding, particularly if accompanied by seasoning, which gives opportunity for all free lime to hydrate before mixing. Failure to hydrate quickly may also be due to too much magnesia, for magnesia, especially after burning, hydrates very slowly. It is mainly for this reason that the amount of magnesia is limited to 5 per cent. An excess of sulphur is also thought to cause unsoundness, by the formation of a compound which crystallizes slowly. Sulphur, however, by retarding the set, gives opportunity for more complete hydration before setting, so that a small amount is allowable and even beneficial.



Specifications require that a "pat of neat cement shall remain firm and hard, and show no signs of distortion, checking, or disintegration, in the steam test for soundness." The steam test is what is called an "accelerated test," because its object is to obtain,

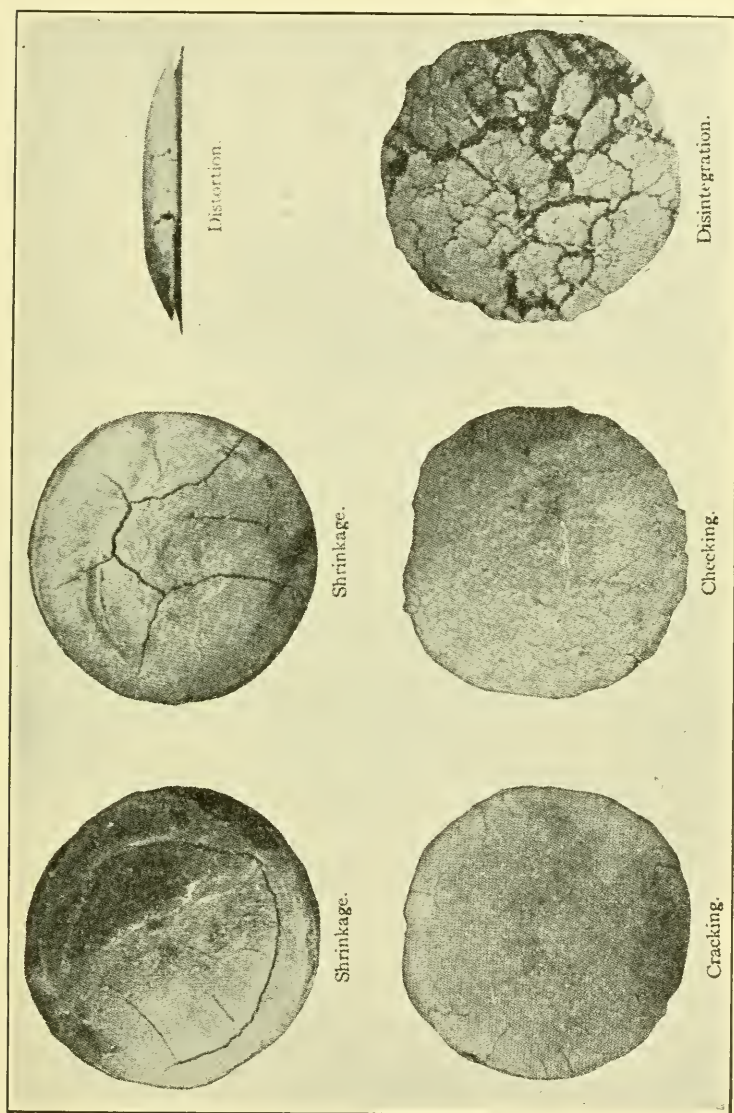


FIG. 45.

tegration, in the steam test for soundness." The steam test is what is called an "accelerated test," because its object is to obtain,

by a short test, the same results that could only be obtained by a very much longer test under practical conditions, since cement or concrete would not in practice be exposed to steam.

The steam test of the A.S.T.M. is described as follows:

A pat from cement paste of normal consistency about 3 inches in diameter,  $\frac{1}{2}$  inch thick at the center, and tapering to a thin edge, shall be made on a clean glass plate about 4 inches square, and stored in moist air for 24 hours. In molding the pat, the cement paste shall first be flattened on the glass and the pat then formed by drawing the trowel from the outer edge toward the center.

The pat shall then be placed in an atmosphere of steam at a temperature between 98° and 100° C. upon a suitable support 1 inch above boiling water for 5 hours.

Should the pat leave the plate, distortion may be detected best with a straight-edge applied to the surface which was in contact with the plate.

Figure 45 shows the appearance of pats that fail in the soundness test.

Formerly it was also required that pats should be observed for soundness, one after 28 days in moist air, and one after 28 days in water at 70° F. The U. S. Government specifications still require that three pats be tested; they are all kept in moist air for 24 hours, then one is kept in air and a second in water at about 70° F. for 28 days, while the third is exposed to steam for 5 hours; if it fails to meet the steam test, the cement may be rejected, or retested after 7 or more days. There has been much discussion and difference of opinion regarding the steam test, because some cements remain sound in air or water but not in steam, while in other cases the reverse is true.

**13. Chemical Composition.**—Specifications require that the following limits shall not be exceeded:

	A.S.T.M.	U. S. Gov't
Loss on ignition, per cent. ....	4.00	4.00
Insoluble residue, per cent. ....	0.85	1.00
Sulphuric anhydride (SO <sub>3</sub> ), per cent. ....	2.0	1.75
Magnesia (MgO), per cent. ....	5.0	4.0

A greater loss on ignition would suggest underburning or adulteration by calcareous material, and a greater insoluble residue might mean improper materials or adulteration by siliceous material. The significance of SO<sub>3</sub> and of MgO has already been alluded to.

Three per cent of  $\text{Ca SO}_4$  gives about 1.75 per cent  $\text{SO}_3$ , so that these limits are usual.

The precise chemical composition desired depends upon the precise silicates or aluminates to be formed; and as there are several of these in which the lime, silica and alumina are in different proportions, all of which set, though some better or more quickly than others, it follows that the proportion of elements may vary considerably without materially affecting the excellence of the cement. A normal American Portland cement which meets the standard specifications has usually a composition within the following limits:

	PER CENT
Silica ( $\text{SiO}_2$ ).....	19-25
Alumina ( $\text{Al}_2\text{O}_3$ ).....	5-9
Iron oxide ( $\text{Fe}_2\text{O}_3$ ).....	2-4
Lime ( $\text{CaO}$ ).....	60-64
Magnesia ( $\text{MgO}$ ).....	1-4

A good cement, however, may have a composition outside the above limits; and a cement within these limits may be poor. Defective cement more often results from imperfect manufacture than from faulty composition.

Based on the desire that certain definite silicates and aluminates should be formed in burning, various chemists have proposed limits for the various constituents. Thus Newberry, an eminent cement chemist, assuming that the tri-silicate ( $3\text{CaO}$ ,  $\text{SiO}_2$ ) and the di-aluminate ( $2\text{CaO}$ ,  $\text{Al}_2\text{O}_3$ ) were the most basic compounds which could exist in good cement (*i.e.*, the compounds with the greatest proportion of lime) gave the formula:

Max. per cent  $\text{CaO}$  = 2.8 times the per cent  $\text{SiO}_2$  + 1.1 times  
the per cent  $\text{Al}_2\text{O}_3$ ,

or, for the cement the composition of which is given in Art. 7

$$\text{CaO} = 2.8 \times 21.08 + 1.1 \times 7.86 = 67.67$$

so that this cement would have less lime than needed to use up all the silica and alumina in producing the compounds named.<sup>1</sup>

<sup>1</sup> "The Constitution of Hydraulic Cements"; N. Y. Section, *Soc. of Chemical Industry*, Oct., 1897. Though Newberry assumed that the dialcic aluminate should be taken, some chemists think the tricalcic aluminate ( $3\text{CaO}$ ,  $\text{Al}_2\text{O}_3$ ) is the most effective in setting. If this is true, Newberry's formula would become  $\text{CaO} = 2.8\text{SiO}_2 + 1.65\text{Al}_2\text{O}_3$ .

The tricalcic aluminate, in hydrating, agglomerates into balls "which hydrate on the exterior to hard masses which prevent the penetration of

The so-called "hydraulic index" is  $\frac{\text{SiO}_2 + \text{Al}_2\text{O}_3}{\text{CaO}}$ , or in the above example  $\frac{28.94}{63.68} = 0.45$ . This hydraulic index has been used for classifying cements, the smaller the index the less the hydraulic activity. But this is defective, for it does not allow for magnesia or iron oxide, and assumes that silica and alumina are equivalent in giving hydraulic activity, which is not the case. This defect has led Eckel to use the "cementation index";

$$\text{Cementation Index} = \frac{2.8\text{SiO}_2 + 1.1\text{Al}_2\text{O}_3 + 0.7\text{Fe}_2\text{O}_3}{\text{CaO} + 1.4\text{MgO}}.$$

He says that the ideal index for a Portland cement is 1.00.

Such indices, however, not only involve assumptions that are as yet unproved, but they can only be used as rough guides, because the action of a cement depends not only upon its composition but more on the conditions of its manufacture. It makes little difference whether a cement has just the proper chemical composition if it is underburned, or overburned, or if the materials have not been sufficiently mixed before burning.

**14. Mortars.**—Neat cement is seldom used except for filling the edges of mortar joints in stone masonry (pointing). It is for other purposes always mixed with sand or some similar fine material to form a mortar. The composition of the mortar is expressed by the ratio of cement to sand, as 1:3. The proportion is in practice by volume, but in the standard tests it is always by weight. As the weight of a given volume depends on its compactness, amount of moisture, etc., these factors should be standardized for testing; and in practice it is customary to specify a stated volume of sand for each bag of cement.

A mortar will be strongest when it is most dense and when the cohesion of the cement and its adhesion to the sand grains, are greatest. The theory of a mortar is that the wetted cement shall completely fill the voids in the sand, and shall completely coat each sand grain.

The voids in a mass of equal spheres, so piled to make the voids a minimum, are 26 per cent of the volume, and this is inde-

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water to the interior; consequently, large masses of unhydrated material are present" (BATES and KLEIN).

No doubt the tricalcic aluminate, the dicalcic aluminate, the tricalcic silicate, and the dicalcic silicate all play a part. The hydration may set free an excess of hydrated lime, which, if it can be dissolved or washed out, may lead to disintegration.



pendent of the size of the spheres. Sand grains are not spherical, nor of equal size, and cannot in practice be piled to produce this percentage of voids. The actual voids will be greater, sometimes up to 50, and averaging perhaps 30 to 40 per cent. If the cement is to coat each sand grain, the latter must not touch, but must be spread apart; hence the volume of cement paste must exceed the volume of voids in the sand if piled alone.

The surface area to be coated will be greater the smaller the grains. In a mass of equal spheres, the total surface area would be doubled if the diameter of grains were halved; the surface varies inversely as the diameter.

For these reasons, with a given proportion of cement, a coarse sand will give a stronger mortar than a fine sand. The greatest strength and economy will be obtained with a sand of mixed sizes. If the sand were graded so that the next to the largest size would just fit into the voids in the largest size, and so on down, the percentage of voids would be least.

The actual percentage of voids in any sand which is to be used should be determined, and the ratio of cement based upon it. The voids should not be found by pouring water into a vessel containing a known volume of sand, because some air will be retained in the sand; but by means of the specific gravity (see Taylor and Thompson, page 165) or by dropping the sand into a given volume of water.

**15. Strength of Cements and Mortars.**—The strength measured is either the tensile or the compressive strength. The standard test is of the tensile strength, notwithstanding that cement or mortar is never used in tension, or, if it acts in tension, its tensile strength is not relied on because, on account of shrinkage due to contraction in setting or to temperature changes, fine cracks are always possible. It has been assumed that the tensile strength gives at all events a measure of the excellence of the cement and its ability to resist compression, although there is no constant ratio between tensile and compressive strength. A test of compressive strength would be more logical.

The form of test piece for tensile tests ("briquette") is shown in Fig. 46, designed so that the clips may grip the ends. Figure 47 shows an automatic testing machine. The upper bucket is filled with shot till it balances the weight on the right, the specimen is placed in the clips, then the shot is allowed to run out, when the excess weight on the right pulls the specimen, through



the lever system. When the briquette breaks, the flow of shot is automatically stopped, and from the weight which has flowed out the tensile pull is determined. If the clips do not grip the specimen centrally, or if the pulls are not in the same line, there is an eccentric load.

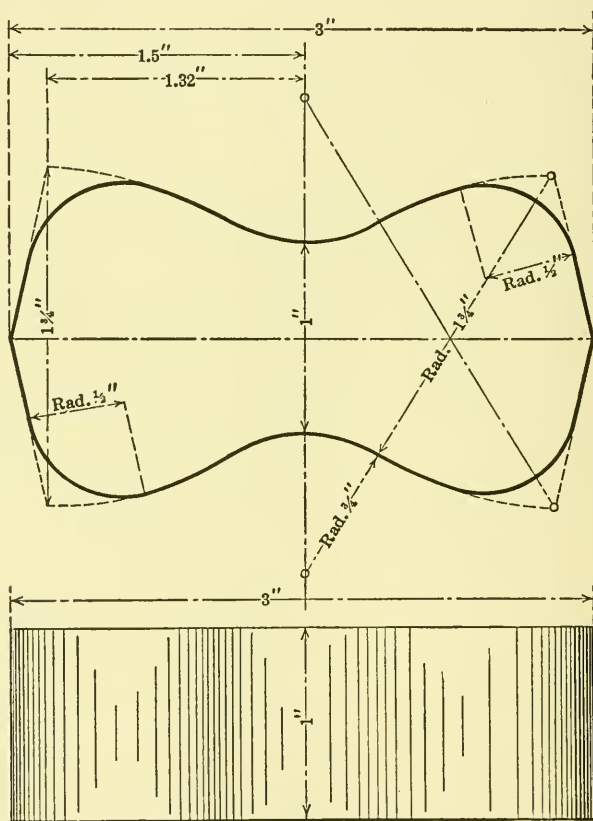


FIG. 46.—Cement briquette.

The ultimate strength will depend on many circumstances, such as the character of cement and sand, the consistency, the manipulation in mixing, the age, etc. All these things are therefore standardized.<sup>1</sup> The standard sand is one from Ottawa, Ill.,

<sup>1</sup> See "Standard Specifications for Cement," *A.S.T.M. Standards*, p. 530, 1921.

furnished by the Ottawa Silica Co., screened to pass a No. 20 sieve and to be retained on a No. 30 sieve.

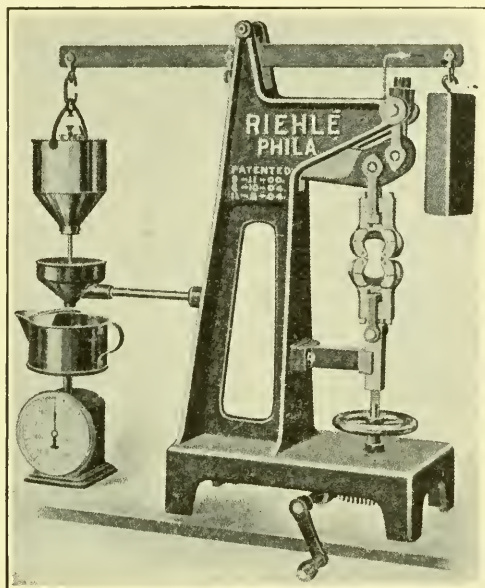


FIG. 47.—Riehlé cement testing machine.

The A.S.T.M. specifications require the following strength:

Age at test, in days	Storage of briquettes	Tensile strength, pounds per square inch
7	1 day in moist air, 6 days in water.....	200
28	1 day in moist air, 27 days in water.....	300

The strength specified is the average of three briquettes of composition 1 cement to 3 standard sand. Formerly neat cement briquettes were also tested, and these are still required by the Gov't specifications,<sup>1</sup> which prescribe:

	POUNDS PER SQUARE INCH
Neat cement 7 days (1 in air, 6 in water).....	500
28 days (1 in air, 27 in water).....	600
1 : 3 mortar 7 days (1 in air, 6 in water).....	200
28 days (1 in air, 27 in water).....	275

<sup>1</sup> The U. S. Gov't specifications, which differ in some respects from those of the A.S.T.M., are published as *Cir. No 33*, of the Bureau of Standards.

It is also required that the average strength of mortar briquettes at 28 days shall be greater than that at 7 days.

The reason why the neat test has been given up by the A.S.T.M. is probably that cement is never used neat (see above) where strength is required, and that the neat test gives no indication of the strength in mortars, which depends upon adhesion.

The detailed treatises give full information regarding the strength of cements and mortars, and their variation with the various factors which affect it.

**16. Concrete.**—Concrete is a mixture of mortar with a “coarse aggregate,” consisting of broken stone, gravel, slag, or cinders. It is thus a mixture of cement with a fine aggregate (sand or stone dust) and a coarse aggregate. The fine aggregate may be considered to be the particles less than  $\frac{1}{4}$  inch in diameter, or which will go through a sieve with  $\frac{1}{4}$ -inch openings; the coarse aggregate, that which is coarser than this. The coarse aggregate is sometimes screened to less than  $1\frac{1}{2}$  inches. The proportions are generally stated by volume, as 1:3:5, meaning 1 volume of cement, 3 of fine aggregate and 5 of coarse aggregate; or the number of cubic feet of sand and of stone per bag of cement are stated. The limiting sizes of stone must be specified.

The theory of concrete is similar to that of mortar. There should be mortar enough to fill the voids of the large aggregate and to coat all the pieces of it. The greatest strength and economy will result, when there is the least cement to secure this result, since cement is the most costly ingredient; and will be when the aggregate is graded so that small stones occupy, as fully as possible, the voids between the larger stones.

**17. Proportioning Concrete.**—Clearly the strength and economy are vitally dependent upon proper proportioning of the ingredients, which are: cement, sand (or fine aggregate), coarse aggregate, and water. The object is to make the most dense concrete with the least cement consistent with the principles above stated.

*Proportioning by Voids.*—This method consists in measuring the voids in the dry materials, supplying cement enough to fill the voids in the sand (with a slight excess) and mortar enough (*i.e.*, sand enough) to fill the voids in the coarse aggregate (with a slight excess). This method is uncertain, because the voids in the concrete will not be the same as in the same dry materials, since the stones will wedge each other apart in a way that is

uncertain, and since the particles must be separated in order that the cement may coat them. Nevertheless, with good judgment and experience, this method gives good results, and large amounts of the best concrete have been proportioned in this way, or even by arbitrarily assigning the proportions according to the average voids in the materials and the character of the work and strength desired.

*Proportioning by Maximum Density.*—This may be done by making trial mixes before the work is begun, and finding the one which gives the smallest volume for a given volume of the unmixed materials.

This may be facilitated by a *mechanical analysis* of the aggregate, which is made by passing it through a series of sieves from the coarsest at the top to the finest at the bottom, and finding the percentage by weight smaller than each given diameter as fixed by the openings between the wires in the sieves. A curve is drawn having for abscissas the diameter in inches, and for ordinates the total percentage smaller than each diameter. William B. Fuller determined, by tests, that the best material was one whose mechanical-analysis curve was a parabola; and from it he was able to tell in what proportions to mix different sizes of materials, which had been separated by screens so that the particles all lay between certain limiting diameters.<sup>1</sup>

*Amount of Water.*—The amount of water should be sufficient to give all that is needed for proper hydration of the ingredients, and to wet all the surfaces of the aggregate. Any deficiency below this, or any excess above, weakens the concrete; for if there is a deficiency the desired hydration will not completely take place, and if there is an excess the density may be decreased. If the concrete is plastic enough, however, the parts will settle into position, and will expel the surplus water, which will rise to the surface, so that it will not form a void of appreciable size; but the water which makes the cement wet must at first exist in the form of small quantities between the particles of the aggregate or cement, which are later absorbed in the hydration, and if in excess must leave small voids. The quantity of water necessary for complete hydration varies with the composition and the precise

<sup>1</sup> See FULLER, WILLIAM B.; Chap. X in TAYLOR and THOMPSON 3d edition; also "The Laws of Proportioning Concrete," *Trans. A.S.C.E.*, vol. 59, p. 67; also RAFTER: "On the Theory of Concrete," *Trans. A.S.C.E.*, vol. 42, p. 104.

compounds formed by the burning. An excess of water is better than an equal deficiency, but the excess should be as small as possible. Rich mixtures require less water than lean ones, because there is less aggregate and therefore, less surface to be moistened, that is, less water is required to produce the desired workable consistency. Professor Duff A. Abrams has studied this matter thoroughly, and thinks that the strongest concrete is not necessarily that having the greatest density, but is that in which the ratio of water to cement, by volume, is the least which will give the proper consistency. Very wet and *sloppy* concretes should be avoided, and also very dry concretes which require excessive tamping. In this, as in everything else, the golden mean is the best. The percentage of water giving maximum strength would give a concrete that would be too stiff to be workable. There must be an excess of water. Concrete should be kept moist after placing, until fully set; it should not be allowed to dry out. This is exceedingly important.

In order to obtain the desired consistency, and uniform consistency throughout, when the same proportions of solid materials are used, the *slump test* or the *flow test* is used. In the slump test a frustrum of a cone 12 inches high, open top and bottom, 8 inches in diameter at the bottom and 4 inches at the top, is placed on a non-absorbent surface, and filled with the concrete in four layers, in a standardized manner. Three minutes thereafter, the mold is lifted off, and the slump is the amount that the height of the concrete decreases. The wetter the consistency, the greater the slump. In the flow test, a frustrum of a cone of concrete is made, the mold lifted off, and the table raised and dropped about  $\frac{1}{2}$  inch fifteen times in 10 seconds; the increase in diameter of the base measures the flow, or consistency.<sup>1</sup>

Professor Abrams has introduced a term, "fineness modulus," which is the sum of the percentages coarser than each sieve, divided by 100: the sieves used being the Tyler standard sieves 100, 48, 28, 14, 8, 4 and the coarser sieves with openings  $\frac{3}{8}$ ,  $\frac{3}{4}$  and  $1\frac{1}{2}$  inches. Thus, suppose the percentages held on the various sieves are as in the table:

<sup>1</sup> It has been suggested to use the flow method, when standardized, to determine time of set. It is obvious that the Gillmore needle tests only the crust. See DAVIS in *A.S.T.M.*, p. 995, 1921.



Sieve	Size of opening	Per cent held on sieve	Per cent coarser than each sieve	If 25 per cent broken smaller
1.5 in.	1.5	0	0	0
0.75 in.	0.75	25	25	0
0.375 in.	0.375	41	66	66
No. 4	0.185	29	95	95
8	0.093	5	100	100
14	0.046	...	100	100
28	0.0232	...	100	100
48	0.0116	...	100	100
100	0.0058	...	100	100
		100	6.86	6.61
			= Fineness modulus	

The *fineness modulus* is 6.86. This modulus gives an indication of the fineness of the material; the larger the modulus, the coarser the material. This will be obvious when we observe that if in the above analysis the 25 per cent held on the 0.75 inch sieve were broken up so that it would all go through this sieve but be held on the one below, the column would read as in the last column, and the modulus would be 6.61. The coarser the aggregate, the smaller the total surface to be wet, and the smaller the amount of water required.

Professor Abrams concludes that the sieve analysis of the aggregate may vary greatly without affecting the strength of the concrete, that the proper basis for proportioning the aggregate is the fineness modulus, and that "the size and grading of the aggregate and the quantity of cement are no longer of any importance except in so far as these factors influence the quantity of water required to produce a workable mix."<sup>1</sup> This does not mean that the quantity of cement is of no importance, for, of course, the cement must fill the voids of the aggregate and surround each particle of it. But *with any given ratio of cement to total aggregate* it is clearly possible to grade the aggregate so as to produce a minimum of voids, and a maximum strength. Assuming that the material of the aggregate is stronger than the cement alone, it is easy to see that the greatest strength in compression

<sup>1</sup> See ABRAMS, D. A.: "Design of Concrete Mixtures," being *Bull. 1 of the Structural Materials Research Laboratory*, Lewis Institute, Chicago, 1919.

may occur with more of the stronger material (aggregate) rather than with the maximum density, and this is what Professor Abrams has found. It would not be true in tension. Professor Abrams assumes a given ratio of cement to total aggregate, and shows how to proportion the aggregate to produce maximum strength.

*Proportioning by Surface Areas.*—L. N. Edwards advises the proportioning of mortars and concretes by surface areas of aggregates,<sup>1</sup> based on the principle that “the physical properties are primarily dependent upon the relation of the volume of cementing material to the surface area of the aggregates.” In other words, his tests indicated, with sufficient closeness, that all mixtures having the same ratio of cement to total surface would have the same strength. (But Professor Abrams does not find this to be true in his tests.) The surface areas were found by counting the number of particles per gram, and from the specific gravity determining the average size. Assuming, then, a sand and a coarse aggregate of given mechanical analysis, and assuming a given area of surface for one gram of cement, the total surface area is estimated and the proportions of sand and coarse aggregate are found, by means of curves and tables.

The main point of the recent studies has been the importance of the ratio of water to cement. Assuming a given consistency, which should be as stiff as the nature of the work permits, and assuming also a given ratio of cement to total aggregate, the grading of that aggregate will clearly determine the strength; and, in compression, the larger and coarser the aggregate (that is, the greater the fineness modulus) the greater in general will be the strength, at least up to a certain limit. This is easily seen by considering one particular large piece of broken stone or gravel aggregate; if this were broken up into a number of small pieces within the same exterior surfaces (discarding some pieces necessarily) there would be some voids within a surface where previously there were none, and the total surface would also be increased; hence, with the same total cement in the mass, if the voids were just filled before, they would not be filled afterward, and the strength would be decreased. An improvement in grading the aggregate may double the strength. The greater the fineness modulus, the larger, in general, the percentage of voids and the less the total surface area; hence, the richer the mix (*i.e.*,

<sup>1</sup> *A.S.T.M.*, pt. II, p. 235, 1918; also YOUNG, in *A.S.T.M.*, pt. II, p. 444, 1919.

the larger the ratio of cement to total aggregate) the larger the fineness modulus for the greatest strength.

It is thus obvious that in proportioning concrete, voids, fineness modulus, surface area, and proportion of water, all come into consideration.

The Joint Committee on Standard Specifications for Concrete and Reinforced Concrete,<sup>1</sup> in its tentative report of June 4, 1921, has made proportioning definite by giving tables of proportions to be used. Knowing the size of the coarse and fine aggregates, and the consistency as shown by the slump test, the table gives the proportions of cement, fine aggregate, and coarse aggregate required to produce a concrete having any desired compressive strength at 28 days, from 1,500 to 3,000 pounds per square inch. The reader should study the report of this Committee, and the other references given in this chapter.<sup>2</sup>

**18. Strength of Concrete.**—Concrete is assumed to act only in compression and shear. Its tensile strength, though not negligible, is ostensibly disregarded on account of the fact that there may be shrinkage cracks; yet its tensile strength is not and cannot be entirely neglected, because where there is pure shear there must be tension, and if shearing strength is depended upon (except where it is merely an oblique shear accompanying compression), so must also tensile strength be depended upon.

The strength is very variable, depending upon composition, manipulation, character of aggregate, and age. The first Joint Committee, in its final report of 1916, recommended as maximum ultimate values to be used in design, for *compression*:

ULTIMATE COMPRESSIVE STRENGTH OF DIFFERENT MIXTURES OF CONCRETE  
(Pounds per square inch at 28 days)

Cement: aggregate.....	1:3	1:4.5	1:6	1:7.5	1:9
Granite, trap rock.....	3,300	2,800	2,200	1,800	1,400
Gravel, hard limestone and hard sandstone.....	3,000	2,500	2,000	1,600	1,300
Soft limestone and sandstone....	2,200	1,800	1,500	1,200	1,000
Cinders.....	800	700	600	500	400

The allowable stresses were given as percentages of the above ultimate values.

<sup>1</sup> This Committee was organized in 1920, as successor of a similar committee organized in 1904, which presented progress reports in 1909 and 1912, and a final report in 1916. It represents five technical societies. Its report of 1921 was published in the *Proc. A.S.C.E.*, Aug., 1921.

<sup>2</sup> See also TALBOT, A. N.: "Strength and Proportioning of Concrete." *A.S.T.M.*, p. 940, 1921.

Professor Abrams found the compressive strength of cylinders 6 by 12 inches, composition 1:5, at 28 days, stored in damp sand, to average 2,990 pounds per square inch when of the best consistency, with a variation from 2,680 to 3,300. The mechanical analyses were very different, but the fineness ratio of the aggregate was constant at 6.04. The surface area per gram of cement varied from 5.6 to 31.3 square inches, averaging 13.8.

With a given ratio of cement to total aggregate, the compressive strength will increase with the fineness modulus up to a certain point, and with the maximum size in the aggregate.

The strength of a concrete may be increased by using more cement, or by improving the grading of the aggregate. With this should go a reduction in the amount of water per unit of cement.

Probably the question of an excess of water has something to do with the relative amounts of crystalline and amorphous hydrates formed, and affects the strength in this way, though the relation does not seem to have been worked out.

*Tensile Strength.*—The ratio of compressive to tensile strength is not constant, but increases with age and with the percentage of aggregate. As a rough average the tensile strength may be taken as one-tenth the compressive strength.

*Shearing Strength.*<sup>1</sup>—The shearing strength is difficult to determine, and tests are at variance. Pure shear, without normal stress, is always accompanied by pure tension of the same intensity on a plane at 45°, so that the pure shearing strength at a given point cannot exceed the pure tensile strength. Talbot<sup>1</sup> found the shearing strength of 1:3:6 concrete at 60 or 61 days, stored in air to be 679 pounds per square inch; and at 61 to 69 days, stored in water, to be 729 pounds per square inch. He says "It appears that the shearing strength is, in general, at least 50 per cent of the compressive strength, and that it may exceed 75 per cent. Evidently the shearing strength of concrete is several times its tensile strength." The apparent discrepancy here is because in tests the average stress over a plane surface is obtained and not the stress at any one point. In tension over a plane surface, a particle of aggregate lowers the strength, because only the adhesion acts, and this is less than the tensile strength of the cement; while in shearing, a particle of aggregate greatly increases the strength, because the shearing strength of the stone is large.

<sup>1</sup> See *Bull.* 8, University of Illinois Expt. Station, 1906.

*Bond, or Adhesion.*—In reinforced concrete it is very important that there should be adhesion between the concrete and the rods which take the tensile stress. These rods are in some cases plain round or square rods, and in other cases the surface is deformed, by ribs or projections, to make it more difficult to pull the rod out of the concrete, or, in other words, to give more bond. The bond per square inch of surface of rod varies according to the shape of the rod, the character of the concrete, the age, and the manner in which the concrete has been placed around the rod.<sup>1</sup> Talbot found for plain round rods a bond of 350 to 450 pounds per square inch at 60 days, higher for rich concrete than for lean (as would be expected); the concrete being of broken stone. The bond will be better the more closely the concrete settles or is rammed around the rods, that is with a wetter concrete than would be desirable for maximum strength of the concrete alone. The Joint Committee report of 1921 allows a *working* bond stress for plain bars of 0.04, and for deformed bars 0.05, of the *ultimate* compression at 28 days, or respectively one-tenth and one-eighth of the working compression in beams; hence, with the same factor of safety in both cases, it assumes the ultimate bond stress for plain bars one-tenth, and for deformed bars one-eighth of the ultimate compression.

Tests of bond strength are made by imbedding the rod in a prism of concrete and pulling it out, by pushing on the prism; or by pushing the rod out (or starting it); or by pulling a rod by beam action, the rod being exposed along its length except at each end, where it is imbedded in the concrete. By the first method, the compression on the face of the block causes a lateral expansion according to Poisson's ratio, which makes the concrete grip the rod more firmly than it would do under ordinary conditions, so that the bond strength found is greater than its true value. Most bond tests, however, are by this method. Withey<sup>2</sup> made tests by the third method, measuring the stress in the rod by extensometers; and also by the first method. The ratio of the average bond by the first method (cylinder) to that by the beam method was from 1.42 for plain  $\frac{3}{8}$ -inch rods to 2.99 for 1-inch rods. Professor H. C. Berry<sup>3</sup> made tests by all three methods, but

<sup>1</sup> See *Bull.* 8, University of Illinois.

<sup>2</sup> *A.S.T.M.*, p. 467, 1908.

<sup>3</sup> *A.S.T.M.*, p. 497, 1909.



modified the beam method so as to avoid finding the stress in the rod by extensometers, by merely cutting the beam into two halves at the center of the span and placing a hinge near the top, so that the stress in the rod was the moment divided by the distance from the rod to the hinge.

**19. Change of Strength with Age.**<sup>1</sup>—We have seen that concrete *sets* in a few hours, and continues to *harden* afterward,

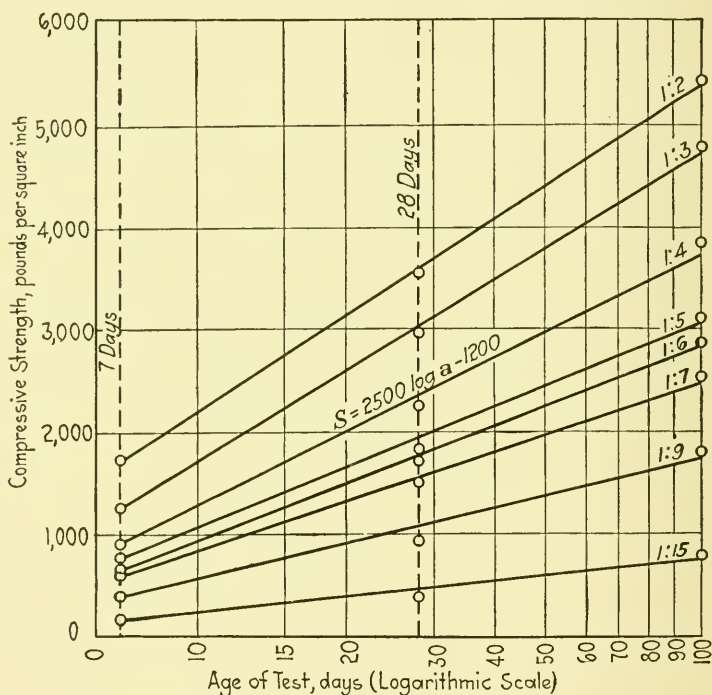


FIG. 48.—Compression tests of concrete (*Lewis Inst. Tests, 6 × 12 inch cylinders, Series No. 93.*)

Abrams on Strength of Concrete, *Am. Soc. for Testing Materials*, vol. XVIII (1918) part II, pp. 324, 329.

probably for a long time. The set is due to the hydration of the constituents, the aluminates setting first. The hardening is due to progressive crystallization, to hardening of the amorphous material, and very likely to the gradual crystallization of material originally amorphous. The strength should increase continuously, but not at the same rate, without retrogression. Tensile strength increases up to 7 days, and more slowly to 28

<sup>1</sup> See a most interesting paper by Professor D. A. ABRAMS, on "The Effect of Age on the Strength of Concrete"; *A.S.T.M.*, p. 318, 1918.

days or longer. It has generally been concluded, from the usual tensile tests, that there is a retrogression in strength from this point, followed by a later maximum. Professor Abrams' studies, however, lead to the conclusion that this is due to the form of the tension test specimen and the method of loading, and that the real strength continues to increase indefinitely, so long as the concrete does not dry out. Compression tests show this continuous increase, without retrogression, up to the longest period covered

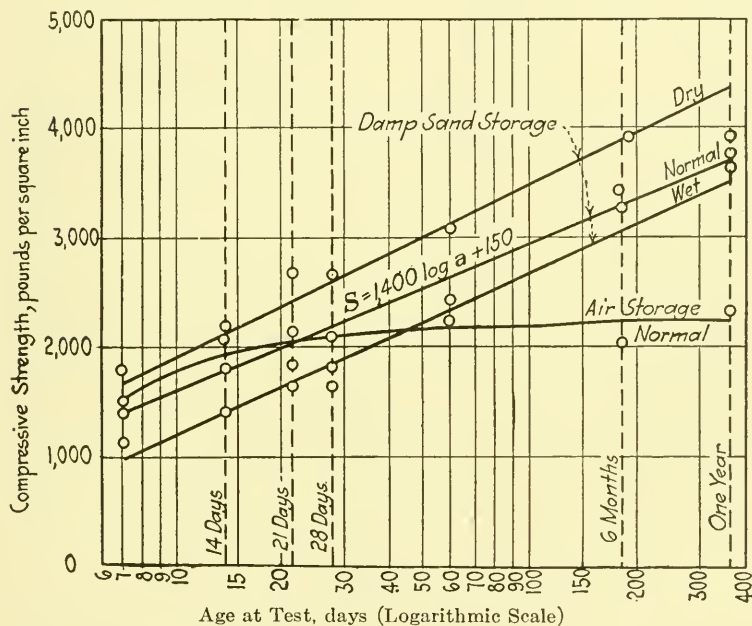


FIG. 49.—Compression tests of concrete. (*University of Ill. Tests, Jour. Am Concrete Inst., Oct.-Nov., 1914, 6 × 6 inch cylinders, 1:2:4 Mixture.*)

Abrams on Strength of Concrete, Am. Soc. for Testing Materials, vol. XVIII (1918) part II, pp. 324, 329.

(9 years), the relation between age ( $a$ ) and strength ( $S$ ) being of the form  $S = n \log a + k$  where  $n$  and  $k$  are constants. It is very important, in order that the strength may increase, that the concrete should be kept wet, which suggests that the increased strength is due to a slow progressive hydration. If the concrete dries out, there is no further increase of strength.

The explanation given by Professor Abrams for the apparent retrogression shown by tensile tests is that on account of the form of the briquette (Fig. 46) the stress is not uniformly distributed

over the smallest section, but is greatest at the edges, as has long been recognized; and that at early ages, when the modulus of elasticity is low, there is more opportunity of adjustment of stress, and hence a smaller ratio of maximum to mean stress, than at later ages when the modulus of elasticity is higher. The tensile test gives only the mean stress, and it follows that as the age increases, though the mean stress may diminish, the maximum stress may increase. This is obviously sound. Figures 48 and 49 show curves of increase of strength.

Unwin gave the formula  $S = a + b(A - 1)^m$  where  $S$  is the strength,  $A$  the age in weeks,  $a$  the strength at seven days,  $b$  and  $m$  constants for any given cement. He found from Grant's tests for neat Portland cement;  $S = 363 + 48\sqrt[3]{A - 1}$ ; for cement mortar, 1:1,  $S = 157 + 40\sqrt[3]{A - 1}$ . He believed, however, that a retrogression occurred after a certain maximum was reached.

**20. Regaging or Retempering.**—It is usually specified that after concrete has partly set it shall not be remixed and the set broken. It is found, however, that if this is done, provided the remixing is thorough and to a uniform consistency, the concrete will set and will ultimately attain about the same strength as if the original set had not been broken, but at a slower rate. In other words, the crystallization and hardening can still go on. This is in agreement with the facts above stated, that cement continues to hydrate and harden even for very long periods. (The same is true in a way, as we have seen, of steel, where the amorphous dust formed by slipping hardens with time and becomes as hard as the original metal; that is, the elastic limit is raised to the previous stress and the elasticity is restored with time.) At the same time, it is not advisable to break up an initial set, particularly if early strength is desired.

**21. Rodding.**—This means repeatedly pushing a pointed rod into concrete while it is setting. This has been studied by Professor F. E. Giesecke,<sup>1</sup> who finds that, by rodding, the strength may be increased as much as 100 per cent. The effect of rodding is to allow entrapped air and water to rise to the surface, and to compact the concrete; while in tamping, the upper portion only is compacted, and the escape of air and water below may be retarded or prevented. The rodding may be continued long after the initial set has taken place, but the crystallization

<sup>1</sup> *A.S.T.M.*, p. 219, 1920; and p. 1008, 1921.

and the hardening of amorphous material can still continue, as above shown. Professor Giesecke has rodded 1:2:4 gravel concrete every 15 or 30 minutes for 4 hours, at which time the rod would not penetrate more than  $2\frac{1}{2}$  inches; but the specimens had a compressive strength of 4,211 pounds per square inch at 28 days and 4,363 pounds at 3 months. The rodding should, however, not be continued so long that the cavity produced by the rod will not close. The relative increase of strength produced by rodding was greater for lean mixes than for rich, as could be foreseen.

**22. Modulus of Elasticity.**<sup>1</sup>—The modulus of elasticity of concrete must necessarily be very variable, depending upon proportions, age, and other factors. The stress-strain diagram is probably concave towards the axis of strain. The value varies from about 1,500,000 to 5,000,000 pounds per square inch. For reinforced concrete work it is generally taken at one-fifteenth that of steel, or 2,000,000.

**23. Poisson's Ratio.**—This is also very variable. It is not often necessary to use it, but when it is, its value may be taken as 0.08 for a 1:3:6 mix, and as 0.18 for a  $1:1\frac{3}{4}:3\frac{3}{4}$  mix.<sup>2</sup>

**24. Coefficients of Expansion.**—The coefficient of expansion of concrete averages about 0.000006 per degree Fahrenheit, while that of steel averages 0.0000066. The close agreement of these makes it possible to imbed steel in concrete without the danger of disintegration due to differences in expansion under changes of temperature.

Concrete also expands or contracts in hardening. This must be due to the changes which take place in the cement, as the aggregate does not change; consequently, the changes must be less the larger the proportion of aggregate, and greatest in neat cement. Concretes hardening under water expand, and those hardening in air contract, but in less degree, and if kept sufficiently wet should expand as those under water do. Most concrete is allowed to harden in dry air, and often is not kept as moist as it should be; and such concrete will contract, the amount depending on the consistency, the proportions, the temperature, and other conditions.

**25. Aggregates.**—Sand for concrete should be clean and of coarse or mixed sizes. It is not necessary that it should be

<sup>1</sup> See *Bull.* 5, Lewis Institute.

<sup>2</sup> See JOHNSON'S "Materials of Construction," p. 480.

sharp. It may contain up to about 10 per cent loam, which, since its particles are very fine, will help to fill voids, and may in small quantities be beneficial, especially in lean concrete, but not in rich. Small amounts of organic matter, and particularly of tannic acid, may be very injurious. It may be detected by the color test, treating the dry sand with sodium hydroxide (NaOH) and observing the color of the filtrate.<sup>1</sup> The color must not be darker than the standard. Organic matter equal to or greater in quantity than 0.1 per cent of the sand may cause serious weakness or injury.

Coarse aggregate is limited in size according to the structure. Generally pieces that will not go through a 2½-inch ring are rejected. For reinforced concrete, which must settle closely around reinforcing bars, the largest size should go through a 1-inch ring. For mass concrete, as for foundations, larger sizes are permitted, up to so-called *plums* or large stones that can be lifted by one man (one-man stones).

Mica in sand or gravel is very bad, as it is in smooth flakes, to which there is little adhesion, and which increase the voids.

Gravel aggregate gives a denser concrete than broken stone, because of the rounded form of the particles. Since the surfaces are smooth, however, the adhesion to them may not be as good as to broken stone.

**26. Permeability.**—Concrete is often desired to be impervious to moisture. This requires the maximum density. Impermeability is sometimes increased by mixing with the aggregate some inert but very fine material. A small amount of loam is sometimes a benefit. The fine material helps fill the voids, and sometimes, by improving the grading of the aggregate, it increases the strength of mortars and concretes. Hydrated lime is often mixed with cement to make it impervious,<sup>2</sup> up to about 15 per cent of the cement and sometimes more. Generally speaking, since this material is weaker than cement, it reduces the strength, but in lean mortars it may increase it by filling the voids and improving the grading. Such addition of slaked lime or hydrated lime is economical, and is often very effective in making concrete impervious. It should not be done where an excess of free lime

<sup>1</sup> See *Cir. No. 1* of the Structural Material Research Laboratory, Lewis Institute, Chicago, 1917.

<sup>2</sup> See ABRAMS, in *A.S.T.M.*, p. 149, 1920, and p. 294, 1921; also SCHERTZER, in *A.S.T.M.*, pt. II, p. 269, 1922; and LAZELL, in *A.S.T.M.*, p. 418, 1908.



would be injurious, as in sea water. and generally not under water in any case.

In *Circ. No. 30* of the U. S. Bureau of Standards, entitled "Lime: Its Properties and Uses," the following statement is made:

In a series of experiments in which the Portland cement in a mortar was replaced by varying amounts of hydrated lime it was found: (1) That hydrated lime up to 15 per cent (by weight) of the cement does not materially affect the strength of the mortar, even when stored under water; (2) this amount of hydrated lime will materially increase the imperviousness to water of even a 1:5 cement-sand mortar; (3) the addition of hydrated lime increases the plasticity of the mortar and makes it easier to work.

**27. Concrete in Sea Water.**—In sea water, concrete should be proportioned and deposited with special care, as there have been numerous failures. These have been due, (1) to mechanical action, blows, etc., as by floating ice, which have damaged the surface or broken off corners; (2) to perviousness of the concrete, which allows water to enter and afterward freeze as the tide falls, causing disintegration, mainly between the levels of high and low tide; (3) failures in sea water have been often attributed to chemical action of the salts in the water, especially sulphates and magnesia, which have been supposed to unite with some of the constituents of the cement, especially the aluminates, forming sulpho-aluminates of lime and magnesia, which crystallize with much water, expanding, and disintegrating the concrete. Later investigations and experience indicate, however, that the most important disintegrating influences are mechanical action and porosity.<sup>1</sup>

Low alumina cements have often been considered best for use in salt water, and the alumina has been limited to not over 6 per cent. But, on the other hand, some high alumina cements have been used with success, and have even been considered superior, so that this matter must be regarded as doubtful, though Le Chatelier considered cements high in alumina to be bad for sea work. No doubt the most important precaution is to secure a dense and impervious concrete and to use a rich mix, especially for the surface layers, which should be as dense and impervious as possible. Often a special mixture is placed so

<sup>1</sup> See *Technologic Paper 12* of the Bureau of Standards for an elaborate study of this question; also Atwood and Johnson in *Trans. A.S.C.E.* for Aug., 1923, and discussion following.

as to form a skin, say a foot thick, of such dense mixture. But hydrated lime should not in this case be used to promote imperviousness.

**28. Uncertainties of Concrete Construction.**—Concrete construction is fundamentally different from construction of steel or any other usual material, in that the concrete is manufactured on the spot out of the component parts, often by unskilled labor, while steel is a finished product brought to the work for erection, after being fabricated by skilled labor. There are thus greater uncertainties in concrete work, in the choice of materials, proportioning, and preparation. Further, in the testing of cement there are more sources of error, due to the large number of variable elements. Nevertheless, concrete has taken its place as one of the most important engineering materials, and is used in enormous quantities.

**29. Microscopic Study of Concrete.**—Within a few years, the microstructure of concrete has been studied. See a good paper on this subject by Nathan C. Johnson, in *Proc. A.S.T.M.*, p. 172, 1915, which should be read by the student. Further study of microstructure will doubtless help to solve some of the problems of setting, and will shed light on proportioning and other still uncertain questions, just as it has in the case of steel.

## CHAPTER XIII

### CORROSION OF METAL—PAINTS AND VARNISHES

1. The importance of protecting wood and metal in structures from decay and corrosion is obvious. Protective coatings are engineering materials of the highest value, and worthy of just as careful study as those which form the structure itself. It is of little use to design an adequate structure if its permanence is not ensured by every reasonable means.

2. **Protection of Metal.**—Metal is destroyed by corrosion or rusting, which is the formation of ferric hydroxide,  $\text{FeO}_3\text{H}_3$ . Regarding the nature of corrosion, Prof. O. P. Watts, in Chap. XXIX of Johnson's "Materials of Construction," says "The two modern theories of the rusting of iron are the *Acid* and the *Electrolytic Theories*. According to the former the presence of an acid is necessary to the formation of rust, but even so weak an acid as carbonic may serve. The acid causes the metal to dissolve, and the oxygen changes the dissolved metal to rust, thereby liberating the acid, which is then capable of dissolving more metal, and so the process goes on. After several years of careful experimenting by different investigators, it now seems to be established that moisture and oxygen are sufficient for the continued rusting of iron, so that the electrolytic theory of rusting is the one more generally accepted." The details of the subject therefore belong in the field of the chemical engineer, and there is, as yet, much conflict of opinion regarding many matters. However, Friend's experiments seem to prove conclusively that in pure water free from  $\text{CO}_2$ , but containing air, iron does not rust.

Certain points, however, seem clear. It is agreed that for corrosion both moisture and oxygen must be present. Iron will not rust in dry air, nor in water without oxygen. It will rust where there is no  $\text{CO}_2$ , but the presence of this, or of acid fumes, salt water, or acid waters, accelerates corrosion. Segregation or other lack of homogeneity in the metal also promote corrosion, and certain impurities are supposed to have the same effect, such as sulphur and manganese, the latter within certain limits. The

presence of rust itself greatly stimulates further rusting beneath. Nickel steel appears to be more resistant than ordinary carbon steel.

It is commonly supposed that steel rusts more than wrought iron, and wrought iron more than cast iron; but it is probable that some observed differences that seem to justify this view are due to other conditions<sup>1</sup> (see Art. 12).

Iron rusts faster in sea water than in pure fresh water, and still faster in sea water polluted by sewage. Cast iron, after long immersion in sea water, has been found to be converted into a soft, black substance, like plumbago, that could be cut with a knife.

Corrosion seems to be affected by stress, strain, and vibration in a manner that is still uncertain. It is curious that steel rails in service do not rust, while the same rails, laid aside, rust rapidly.

A coating of Portland cement seems to prevent rust. This is fortunate, in view of the extended use of reinforced concrete. It shows the importance, however, of making the concrete in such structures so that a film of cement will surround the steel, by using a rather wet mix, tamping it well, and choosing a bar of suitable shape. Concrete, when new, is alkaline, which prevents rusting. Many believe that in time it becomes neutral, and then protects the steel only mechanically. The protection may be perfect, but is not always so. Many believe that it is best not to paint steel bars to be used as reinforcing, and that it does not seem necessary to remove rust that may have formed before placing. This must be said to be still a doubtful question.

Nevertheless, the question of preventing corrosion of the steel in reinforced concrete is an important one, especially for concrete to be immersed in water, particularly in sea water. Some of the reinforcing bars in concrete viaducts over sea water, which the writer has examined, have been found to be considerably rusted and the concrete protection broken off. Whether the breaking of the concrete exposed the steel and led to its rusting, or whether rusting of the steel caused the concrete to break off, could not be ascertained.

R. A. Cummings states as his practice: "Wherever reinforcement is imbedded in concrete that is submerged in water or subjected to moisture, the rods are completely coated by immersion in a bath of neat Portland cement immediately before being placed in the work."<sup>2</sup>

<sup>1</sup> See *Proc. A.S.T.M.*, p. 247, 1908 and p. 155, 1905.

<sup>2</sup> *Proc. Eng. Soc. Western Pennsylvania*, Jan., 1909.

Galvanizing or painting with oil or tar should of course be avoided as reducing or destroying the adhesion of the concrete.

3. For steelwork exposed to the air, corrosion may be prevented by proper paint. Therefore steel structures should be designed so that every part, if possible, should be accessible for inspection and painting. The structural engineer should know what kind of paint to use and how to use it. The subject is a specialized one, largely a branch of chemistry, but some fundamental principles should be stated.

#### PAINT

4. **Necessary Qualities of Paint.**—The qualities which it is desirable that a paint should possess are:

It should be impervious to moisture.

It should be as impervious to air as practicable.

It should be somewhat elastic, so that it will not crack when the metal deforms under stress.

It should be inert, containing no ingredients that will attack metal.

It should be durable.

It should not be acted on or attacked, after it has dried, by air or any gases likely to be mixed with air.

It should be tough and hard, so that it will resist reasonable abrasion to which it may be exposed.

5. A paint consists of a *base* and a *vehicle*, to which are sometimes added solvents or thinners to make it more liquid, driers to hasten the hardening, and pigments to give the color desired, if not given by the base. The base gives opacity, color, solidity, imperviousness, and hardness; the vehicle gives elasticity, strength, and cohesiveness.

6. The *vehicle* is almost always linseed oil, which, when exposed to the air, hardens, not by evaporation as wet bodies do, but by absorbing oxygen and becoming converted into a tough, elastic material called linoxyn. The *drying* is hastened by dry weather and sunlight, and by the use of so-called "driers," but raw linseed oil generally requires 24 hours and often much longer. Raw linseed oil is either hot-pressed or cold-pressed: cold-pressed oil has a golden-yellow color, while hot-pressed oil is darker and less fluid, and contains some fats and solid organic matters which are harmful.

Linseed oil is either raw or *boiled*. Boiled oil is not really boiled, but merely heated with certain substances, called "driers,"



which are oxidizing agents, such as oxides of lead or manganese, which transfer oxygen to the oil and so make it thicker, and make it dry more quickly, than raw oil. The raw oil, however, is more durable than *boiled* oil, and only small quantities of drier should be used. Boiled oil, however, hardens quicker and makes a harder film; and since water tends to soften any paint film, and the harder film resists this best, boiled oil is preferable for paint exposed to water, as for the interior of water tanks and stand-pipes; and many prefer it to raw oil for all exterior work, notwithstanding its smaller durability. The film given by boiled oil, however, is sometimes too hard, and more liable to crack than the film of raw oil. The greater thickness of boiled oil is often counteracted, and the paint made more workable, by diluting it with *thinners*, chiefly turpentine, which completely evaporates. Raw linseed oil hardens so slowly that often a drier must be added to the raw oil, without heating, while boiled oil often requires a thinner. The danger in a drier is, that since it is an oxidizing agent, it may, if in excess, oxidize the metal, and so contribute to the effect that the paint is supposed to prevent. The danger in a thinner is that the paint may be made so thin that it spreads into too thin a film, thinning the oil which gives it strength, and so does not give sufficient protection or durability. It is for these reasons that some bridge specifications (*e.g.*, Department of Railways and Canals, Canada) prohibit the use of all driers and thinners. Some substances formerly used as driers contained rosin, which was therefore found in some commercial *boiled* oils; but within recent years rosin is not found in commercial boiled oil, and since rosin is injurious because it causes brittleness and lessens durability, it should not be allowed in driers or oils.

Altogether, it seems desirable to use raw oil, with a small, specified, amount of drier which shall contain no rosin. If a harder film is desired, a mixture of boiled and raw oils may be used, with specified small amounts of drier and thinner; and for continued exposure to water boiled oil should be used. *Circular No 69* of the U. S. Bureau of Standards states (page 13) "This Bureau knows of no case in which boiled oil is used in paint, where equally satisfactory results could not be obtained by the use of raw oil and a suitable Japan drier." Owing to the uncertainty of the weather, it is generally necessary to use some drier for out-door painting. Also, while most engineers formerly

believed the use of a volatile thinner undesirable, probably because of the danger of excessive use to make the work easier for the painter, it is now generally believed that a proper amount of such thinner is desirable, particularly if boiled oil is used, and especially for undercoats on wood, which should be dried through before the next coat is put on. The proper amounts of oil, drier, and thinner depend on "the nature of the pigment, character of the surface to be painted, whether undercoat or finishing coat, and exposure (whether inside or outside)." In painting wood, the pores, if not previously filled by a *filler*, will absorb considerable of the vehicle, which should therefore be in greater amount than for painting metal. The use of a thinner will help this absorption. It is often desirable to use turpentine in repainting, to soften the old paint and make the new paint adhere better. It may also be used when it is desired to have much pigment, as it evaporates, and so really reduces the proportion of oil, while keeping the paint workable.

7. The *base* is an inert, opaque, durable material, which is spread over the surface by the vehicle. It should be finely ground, so that one particle may not occupy the whole thickness of the film. The more finely ground, the better the paint will spread. Red lead, white lead, iron oxides, carbon, and other substances, are used. The most widely used is red lead ( $\text{Pb}_3\text{O}_4$ ), which is made by heating litharge ( $\text{PbO}$ ). The more finely ground the litharge, the greater the proportion of red lead in the product, and the finer that product, hence the better the paint. Litharge is easily decomposed, and is acted upon by linseed oil, while red lead is more stable and permanent. The base should not be acted on by the oil. Litharge in oil becomes ropy, and on standing hardens to a solid. If commercial red lead contains much litharge, it means that the litharge was not finely ground, the paint will harden in the can if kept, the paint is heavy, forms lumps, runs, and is hard to apply. The finer the pigment, the more it attracts the oil to bind it into a firm film, and the more it repels water. Formerly commercial red lead contained 10 to 15 per cent of litharge, and while it made a good paint, it had the objections that it dries quicker than pure red lead, but is not so durable. Lately commercial red lead can be obtained which contains 98 per cent or over of pure red lead. Paint made of this high-grade product works easily, and is very permanent. It is put up as a paste in cans, and does not harden.

The U. S. Gov't specifications require red lead for paint to contain at least 94 per cent pure red lead, with a higher percentage in some cases.

Other bases, such as iron oxides, white lead, carbon, graphite, silica, cement, and others, have been used for paints on metals, but the general opinion of engineers is that nothing is so good as red lead. Generally, from 25 to 30 pounds of pure red lead is used to a gallon of oil, though often more red lead is used, as in the Hell Gate bridge, where 37.5 pounds were used to a gallon of oil.<sup>1</sup> The Metropolitan Water Board of Massachusetts has largely used for steel exposed to water a paint of 98 per cent red lead to which is added a small amount of finely powdered litharge, for the purpose of getting a hard coat, using also boiled oil.

**8. Varnish.**—Paint is a mixture; varnish is a solution of some resin or gum in a liquid which evaporates and leaves the resin as a hard and sometimes transparent coating. The liquid is spirit or oil. In an oil varnish the oil forms part of the film. If a pigment is added to a varnish, it is an enamel paint, the varnish being the vehicle. Linseed oil is often used in oil varnish, and Chinese Tung oil in certain cases, which dries more rapidly than linseed, and is tough and durable. Rosin is bad in varnish, as in paint, making the film brittle, causing it to lack durability.

**9. Painting on Wood.**—Wood can absorb some of the paint vehicle. It is therefore necessary first to fill the pores with a *filler*, which may be paste or liquid, the latter being more convenient; or else to proportion the undercoat so that some of the vehicle may be absorbed without injury to the film. Also, paint on wood may not only keep moisture out, but may keep moisture in and so cause decay (dry-rot). Hence paint should be applied only to dry seasoned wood. The best filler is linseed oil, sometimes thinned with turpentine if applied to resinous wood. Knots should be coated with shellac before painting. Common bases for white paint on wood are white lead and zinc white; many think zinc white the best, or a mixture of the two.

**10. Painting.**—A good paint depends for its durability quite as much upon a proper preparation of the surface, and proper methods of application as upon proper material.

Painting should never be done in wet or damp weather or when the temperature is under 50° F. (some say 40° F.).

<sup>1</sup> SABIN, Red Lead p. 28.

In painting wood, the surface should be smoothed by sand-papering, and should be clean and dry.

In painting metal, all rust, dirt, and loose mill scale should be removed, and all grease removed by washing with benzine or caustic soda. The surface should be thoroughly dry and clean. There are three methods of cleaning a metal surface:

(1) By the sand blast. This is a thoroughly efficient process, but expensive. It leaves the surface so clean that it is particularly liable to rust, and it should be painted immediately after cleaning, to secure good results. Some engineers do not like it for this reason, but no matter what method is used, if thoroughly cleaned, the surface should be painted at once.

(2) By pickling, which is immersion in dilute acid, after cleaning of dirt and grease. After pickling, the acid must be removed by washing with an alkali. Pickling is thought by some to be better than sand-blasting, as it leaves the surface rough, and the paint adheres better, while sand-blasting smoothes the surface. Pickling sometimes causes a surface brittleness which is most marked in thin pieces.

Pickling is of course possible only before original construction.

(3) By far the most common method is to clean the scale and rust off by scraping and hammering, and the use of wire brushes. Very adherent mill scale may not be removed in this way, and many think that it does no harm to paint over such scale, while others deny this and claim that, if there is any rust beneath the paint, rusting will continue. It is doubtless better to remove all mill scale, for it may loosen later. Of course all *loose* scale should be removed.

The Committee of the A.R.E.A. on Preparation of iron and steel surfaces for Painting reported in 1921 that scraping, wire brushing, and wiping gave as good results as sand-blasting or pickling where corrosion had not commenced or had only moderately progressed. They also stated that treating old painted surfaces which have bare rusted spots by brushing the coating with benzine, burning the benzine off, then scraping and wire brushing, gave better results than without the benzine treatment (Proc. A.R.E.A., p. 345, 1921).

**11. Number of Coats.**—Steelwork should be given one coat of pure red lead before leaving the shop, though rarely a coat of linseed oil is used instead. The oil or paint film contracts in drying, and hence tends to draw away the film from protuberances like rivet heads and from the edges of angles and other shapes.



Hence Professor Sabin recommends that before putting on the first field coat, all rivet and bolt heads should be painted, all places not well covered by the original shop coat should be repainted, and that all edges shall receive a narrow striping coat extending an inch from the edge. When this is dry a full coat should be given to the entire surface, and, when this is dry, a third coat. No coat should be applied till the previous coat is dry.

It is also considered that wood should originally receive three coats. For repainting, the old paint need not be removed unless loose, and one or two coats may be enough.

The covering capacity of a paint depends upon the character of the surface, the fineness of the base, the consistency of the paint, and the degree of brushing.

The price of steelwork is estimated by the ton, and it is often convenient to estimate what the paint will cost per ton of steel. It is clear that this is an improper basis, for it is not tonnage, but surface, that is painted. A gallon of red lead paint of proper consistency will cover about 700 square feet. A ton of thin material will require more paint than a ton of thick material. Professor Sabin gives tables, and comes to the conclusion that "on medium weight bridges three-eighths of a gallon of paint per ton for the first coat, one-fourth gallon for the second coat, one-fourth for the finishing coat, or seven-eighths gallon for the three coats, would be sufficient; heavier bridges less, and lighter bridges more; roofs 1,000 to 1,200 square feet per gallon."

**12. Steel vs. Wrought-iron.**—Reference has been made in Art. 1 to the common belief that steel rusts faster than wrought-iron. This subject is discussed in the references given. The writer definitely shares in this belief, though not based on scientific demonstration, but on his experience. The wrought-iron links of the suspension bridge at Newburyport, Mass., built in 1810, which was rebuilt in 1910 by R. R. Evans, County Engineer, with the writer as Consulting Engineer, were in very good condition after 100 years of service. On the New York elevated railroads a steel structure for third track was built of steel in 1916, and this had in 1923 deteriorated much more than the wrought-iron structures adjoining, though the latter had not been painted since several years before 1916. This is of course not conclusive, for the paint on the wrought-iron may have been better than on the steel, though there is no reason to suppose it was. Nevertheless, steel is and will be almost exclusively used



for structures, and the question is how best to protect it; and many metallurgists believe that there is little if any difference between good wrought-iron and good steel as regards corrosion. There is good evidence, however, that cast-iron resists corrosion better than wrought-iron or steel.

**13. Copper in Steel.**—There is good evidence that a small percentage of copper in steel greatly reduces the corrosion of uncoated sheets exposed to the weather. The Committee of the A.R.E.A. on corrosion has made many tests at Pittsburgh and in 1921 reported: "We may definitely conclude that copper-bearing metal shows marked superiority in rust-resisting properties in comparison to non-copper-bearing metal of substantially the same general composition, from which superiority we may truly anticipate a marked increase in the service life for copper-bearing metals under *atmospheric exposure of uncoated sheets*" (*Italics ours*). On the other hand, when immersed in mine water the copper had little effect, and, if there was any, its presence gave a slightly shorter life. The Committee, however, thought it "safer not to draw definite conclusions" till the tests should be complete. However, exposed to the atmosphere, of 132 copper-bearing sheets, No. 16 gage, none had failed, while of 126 non-copper-bearing sheets 54 had failed, after 52 months' exposure; and of 146 copper-bearing sheets, No. 22 gage, 93 had failed, and of 84 non-copper-bearing sheets 82 had failed after the same time.

Professor W. H. Walker (Trans. Am. Electro-Chemical Soc. 1921) said:

Impartial evidence is now legion that no commercial iron or steel so well withstands atmospheric corrosion as does steel containing approximately 0.2 per cent copper.

Professor Samuel L. Hoyt of the University of Minnesota, in an article in "*Chemical and Metallurgical Engineering*," Aug. 1, 1919, says:

The general conclusion which it is believed may be drawn from this exposure test is that so-called copper-bearing steel, in which the copper content is about 0.20 to 0.25 per cent, offers the greatest resistance to corrosion of the common sheeting materials. After copper-bearing steel come pure open-hearth iron, open-hearth steel, and bessemer steel in order of excellence. The two best sheeting materials, according to this test, are copper-bearing steel and pure iron. Both as regards resistance to atmospheric corrosion and cost of production, copper-bearing

steel seems to possess a decided advantage over pure open-hearth iron as a sheeting material.

Copper steel with about 0.25 per cent copper (0.2 to 0.3) has been largely used in the construction of steel cars, where corrosion is apt to be rapid, not only on account of the weather, but also on account of the sulphur contained in coal and ore.<sup>1</sup>

It has not been used for bridges, so far as the writer knows, except in the Fortieth St. bridge across the Allegheny river in Pittsburgh, built in 1924, in which copper-steel with about 0.2 per cent copper was used, with an allowed variation of 0.02 above or below.

The additional cost of copper steel of this composition above ordinary structural steel is stated to be about \$3 per ton (Dec., 1923).

**14. Metallic Coatings.**—Steel and iron are often protected by metallic coatings, either (a) of a different metal, or (b) in which the surface metal itself is converted into some less corrodible compound.

Of the coatings of the first class, zinc coatings are considered the best for protection against corrosion. *Galvanized iron* is iron or steel coated with zinc. It is stated that "the life of zinc coatings, particularly those of a porous character, may be prolonged almost indefinitely by periodically oiling them."<sup>2</sup>

Of the coatings of the second class, a well-known one is that produced by the Bower-Barff process, or modifications of it, by which the metal is oxidized to  $\text{Fe}_2\text{O}_3$  by being heated in superheated steam, or in a mixture of steam and benzine. A layer of oxide formed at the temper color of a blue heat, or other color, which forms a layer of oxide, when heated in free air, is often used.<sup>2</sup>

Metallic coatings have little application in structural work, except that galvanized iron is often used for the roof covering and sides of buildings.

**15. References.**—The above will, it is hoped, enable the reader to understand current specifications and the reasons for them. The following authorities should be consulted for further study:

<sup>1</sup> See *Railway Age*, June 16, 1923; "Reducing the Corrosion in Steel Cars," by J. J. Tatum, Supt. Car Dept., B. & O. R. R.

<sup>2</sup> *Circular No. 80*, U. S. Bureau of Standards, on "Protective Metallic Coatings for the Rustproofing of Iron and Steel." This contains a good bibliography.

1. SNOW: "Paints for Wood," Chap. XV, with many references.
2. *Circular No. 69*, U. S. Bureau of Standards, on "Paint and Varnish."
3. SABIN, A. H.: "Technology of Paint and Varnish," John Wiley & Sons, Inc., 1917.
4. SABIN, A. H.: "Red Lead and How to Use It," John Wiley & Sons, Inc., 1920.
5. STOUGHTON: Chap. XVI on Corrosion of Iron and Steel.
6. JOHNSON: Chap. XXIX on Corrosion of Iron and Steel.
7. FRIEND: "The Corrosion of Iron and Steel," Longmans, Green & Co., 1911.
8. FRIEND: "Chemistry of Paints," Longmans, Green & Co., 1910.
9. LANG: "The Corrosion of Iron and Steel," McGraw-Hill Book Co., Inc., 1910.
10. SANG: *Proc. Eng. Soc. West. Pa.*, Jan., 1909, with discussion.
11. *Am. Soc. for Testing Materials*. Various volumes, see Index.

### WATERPROOFING

16. The waterproofing of masonry or concrete is of importance. The materials used are of two kinds; (1) fabrics, or (2) so-called "integral" waterproofing. Waterproofing by fabrics consists in using several layers of tarred felt, paper, or cloth saturated with a waterproofing substance. These are placed on the outside of the *masonry* or concrete, each layer swabbed with coal tar, asphalt, or some such substance, sometimes four or five layers being used, to prevent the water from penetrating the concrete. Integral waterproofing consists in making the concrete as impervious as possible, by adding to it some substance that will fill the pores and keep out water. Sometimes only the outer skin, or the inner skin, is made of this waterproof material. Sometimes, if a cellar wall leaks, the endeavor must be made to make it tight by supplying an inner layer of some waterproof material.

There are a number of compounds, methods, and fabrics, offered for use, and the reader is referred to Sweet's catalogue, to the advertisements in the engineering periodicals, and to the catalogues of advertisers, The Proceedings of the A.S.T.M., the A.C.I., and the A.R.E.A. for many papers on this subject.

As to a comparison of the fabric with integral waterproofing, it is obviously better to keep the water from getting at the concrete than it is to try to make the latter impervious, and the writer, in his many years of experience in constructing subways in Boston, has always used fabric. The Committee of the A.S.C.E. on Concrete and Reinforced Concrete, reporting in 1917, said:

When mortar or concrete is proportioned to obtain the greatest practicable density and is mixed to a proper consistency, the resulting mortar or concrete is impervious under moderate pressure.

On the other hand, concrete of dry consistency is more or less pervious to water, and, though compounds of various kinds have been mixed with the concrete or applied as a wash to the surface, in an effort to offset this defect, these expedients have generally been disappointing, for the reason that many of these compounds have at best but temporary value, and in time lose their power of imparting impermeability to the concrete.

The Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, representing five engineering societies, in its Tentative Specifications submitted June 4, 1921 (see Trans. A.S.C.E.) says. "Integral Compounds shall not be used." Nevertheless, they are useful in certain cases.

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